Table of Contents

30864100179145

Table of Contents
(Continued)

Chapter Nineteen
SUBSTRUCTURES AND BEARINGS

Section 11 of the **LRFD Bridge Design Specifications** discusses the design requirements of abutments, piers and walls. Chapter Nineteen presents MDT supplementary information on the design of these structural components. Section 13.4 of the **MDT Structures Manual** presents Department criteria for the selection of substructure components within the context of structure type selection.

19.1 ABUTMENTS

19.1.1 General

An abutment can include a backwall, a cap and wingwalls. The term "end bent" is often used interchangeably with "abutment." A backwall is the portion of the abutment which functions as a wall providing lateral support for fill material on which the roadway rests immediately adjacent to the bridge.

Abutments can be classified as rigid or flexible abutments. Flexible abutments eliminate joints at the end of the superstructure by integrating the bridge deck with the backwall. Rigid abutments incorporate expansion joints at the end of the bridge between the deck and the backwall to accommodate thermal movements. Flexible abutments must be able to accommodate the movements through elastic behavior of the bridge and the surrounding soil because the deck is integral with the abutment.

An abutment may be designed as one of the following three types in descending order of preference:

1. Semi-integral Abutment. Flexible abutment with a pin joint between the backwall and cap to facilitate construction and subsequent maintenance.
2. Integral Abutment. Flexible abutment without a joint between the backwall and pile cap (in cross section, the backwall and pile cap may, in fact, appear as a monolithic rectangle with no apparent cap).
3. Free-standing Abutment. Rigid abutment with a joint between the bridge deck and the backwall.

Figure 19.1A presents schematics for the three basic types of abutments. Each of these is discussed in this Section.

Abutments shall generally be of the cast-in-place, reinforced concrete type. They shall be founded on spread footings, drilled shafts or driven pile footings.

A jointless flexible abutment, either integral or semi-integral, is preferred. Free-standing rigid abutments shall be used where the anticipated translational movements of the piles are too great, or settlement of the backwall is anticipated. The force effects of these displacements must be included in the design.

19.1.2 Loads

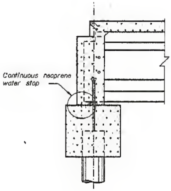
Reference: LRFD Articles 11.6.1.1 and 11.6.1.3

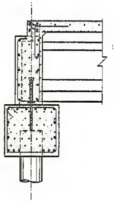
The static earth pressure shall be determined in accordance with Article 3.11 of the LRFD Specifications. Generally, no passive earth pressure shall be assumed to be generated by the prism of earth at the near face of the wall.

19.1.3 General Design and Detailing Criteria

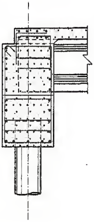
The following applies to the design and detailing of backwalls and wingwalls:

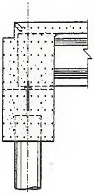
Semi-Integral Abutment



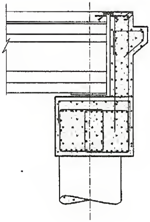


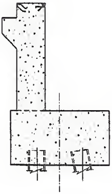
Integral Abutment





Free Standing Abutment





TYPICAL ABUTMENT TYPES

Figure 19.1A

1. Bridge Approach. Typical MDT practice is to design for the future possibility of a bridge approach slab but to not build the slab in the initial construction. When reinforced concrete bridge approach slabs are used, live load surcharge will not be considered on the end bent; however, the vehicular loads on the appropriate slabs shall be considered. Anchor the appropriate slab to the abutments if in a high seismic zone. A rigid approach slab helps to prevent compaction of the backfill behind the abutment.

   Provide a paving notch on all on-system structures and off-system structures that have approach roadways that are paved or likely to be paved. If an approach slab will be constructed, show it on the General Layout.

2. Bridge Approach Joints. Provide a terminal joint or pavement relief joint at the end of the roadway of the bridge approach slab if the roadway pavement is concrete. A joint is not required if the entire adjacent pavement is asphalt.

3. Wingwall Connection. In general, U-shaped wingwalls should not extend more than 3 m behind the rear face of the abutment. If wingwalls longer than 3 m are needed, then an auxiliary footing must be provided. Also, if longer extensions are necessary, force effects in the connection between the wingwall and abutment, and in the wingwall itself, shall be investigated, and adequate reinforcing steel shall be provided.

4. Thickness. The minimum wall thickness for an abutment is 350 mm. Walls may be of constant thickness or with a battered fill face as required. Typically, the near face shall be vertical but, if conditions warrant (e.g., high walls, anticipated tilting), it may be slightly battered.

5. Expansion Joints. Vertical expansion joints should be considered for wall lengths exceeding 30 m.

6. Backwall/Wingwall. The junction of the abutment and wingwall is a critical design element, requiring special considerations. If the wingwall is tied to the backwall (i.e., there is no joint), design for at-rest pressure. All reinforcement must be developed into both elements such that full moment resistance can be obtained.

7. Backwall Batter. Vertical backwalls are preferred (i.e., no batter). For tall, free-standing walls, batter may be considered. Where used, the batter should be between 1:10 and 1:15 (H:V).

8. Backfill. Abutments and wingwalls shall be backfilled with Select Backfill specified by the Geotechnical Section. The neat line limits of the Select Backfill shall be shown on the plans or described in the special provisions. Show the Select Backfill quantity on the road plans.

9. Reinforcing Steel. If an expansion joint is located directly over the abutment cap, all reinforcement in the abutment wall shall be epoxy coated.

19.1.4 Semi-Integral Abutments

The semi-integral abutment, or stub abutment, is MDT's typical end-bent configuration. Transverse and longitudinal superstructure forces are transmitted to the substructure through radius plate steel shoes with anchor bolts that allow rotation. Typically, the backwall and wingwalls are cast around the girder ends, attached to the slab and isolated from the pile cap. When U-shaped wingwalls are used, the wings can either be monolithic with the backwall and isolated from the pile cap or attached to the pile cap with the backwall left free to rotate. The joint between the backwall and the pile cap facilitates raising the superstructure if settlement occurs.

19.1.5 Integral Abutments

Reference: LRFD Article 11.6.2.1

19.1.5.1 General

Traditionally, bridges have been designed with expansion joints and other structural releases that allow the superstructure to expand and contract relatively freely with changing temperatures and other geometric effects. Integral abutments eliminate expansion joints in the bridge decks, which reduce both the initial construction costs and subsequent maintenance costs.

Using integral abutments is effective in accommodating the horizontal seismic forces. Minimum beam seat length requirements need not be investigated for integral abutment bridges.

19.1.5.2 Design Criteria

The following requirements must be satisfied in all cases where integral abutments are used:

1. Backfill. All integral abutments for girder type superstructures shall be back filled with Select Backfill.

2. Steel Girder Stability. Where steel girders are used, an analysis of the non-composite girder stability should be made to locate the first intermediate diaphragm to provide stability prior to and during the deck pour. In lieu of the analysis, an intermediate diaphragm should be placed within 3 m of the end support. An analysis will most likely yield a more economical, larger diaphragm spacing.

19.1.5.3 Superstructure and Interior Sub-Structure Design Criteria

Although the ends of the superstructure are monolithically attached to integral abutments, the rotation permitted by the piles is sufficiently high, and the attendant end moment sufficiently low, to justify the assumption of a pinned-end condition for girder design. The ends of the structures are also assumed to be free to translate longitudinally.

19.1.5.4 Integral Abutment Details

Integral abutments are typically constructed using the following preferred method. The superstructure girders are set in place and anchored to the previously cast-in-place abutment cap. Typically, the concrete above the previously cast-in-place cap is poured at the same time as the superstructure deck. To address steel girder stability, refer to Comment #2 in Section 19.1.5.2.

Optional construction joints may be placed in the cap to facilitate construction. The optional joint below the bottom of beam may be used on all integral abutment bridges regardless of bridge length.

The abutment details shall meet the following requirements:

1. Width. The backwall width shall not be less than 750 mm.

2. Cap Embedment. The piling shall extend a minimum of 500 mm into the cap.

3. Concrete Cover. Concrete cover beyond the farthest most edge of the girder at the rear face of the abutment shall be at least 100 mm. The minimum cover shall also apply to the paving notch area. The top flange of steel girders and prestressed I-girders may be coped to meet this requirement.

4. Girder Anchorage. A minimum of three holes shall be provided through the webs of steel girders and through prestressed I-girders to allow #19 bars to be inserted to further anchor the girder to the cap. Position the holes so that, when the bars are inserted, they will be within the backwall cage.

5. Reinforcement. The minimum size of stirrups shall be #13 spaced at a maximum of 300 mm. Longitudinal backwall reinforcing steel be #22 @ 300-mm maximum spacing along both faces of the abutment.

6. Corner Bars. Use L-bars extending from the rear face of the backwall into the top of the slab at 300-mm spacing or less.

19.1.6 Free-Standing Abutments

19.1.6.1 Usage

Use free-standing abutments where integral and semi-integral abutments cannot accommodate the magnitude of the longitudinal movements. Free-standing abutments can be founded on piles, drilled shafts or spread footings.

19.1.6.2 Epoxy-Coated Steel

For abutments that have a bridge deck expansion joint located between the end of the deck and the face of the backwall, all reinforcing steel in the abutment shall be epoxy coated. This includes all cap, backwall and, if present, wingwall reinforcing.

19.1.6.3 Seismic Shear Blocks

In seismic areas, shear blocks may be formed into the top of the abutment cap to provide lateral restraint for beams that do not have side restraint provided by the bearings or other means.

19.1.7 Pile Spacings and Loads

19.1.7.1 General Design Criteria

The following criteria applies to piling for both integral and semi-integral abutments:

1. Pile Spacing. Use a single row of piles for an integral or semi-integral abutment. Pile spacing should not normally exceed 3 m; however, if the cap is properly analyzed and designed as a continuous beam, this restriction need not apply. If practical, one pile may be placed beneath each girder. To reduce force effects for a large beam spacing, consideration may be given to twin piles under the beam, spaced at not less than 750 mm. See Chapter 20 for minimum pile spacings. The piles are considered to be free ended and capable of resisting only horizontal and vertical forces.

2. Number. The number of piles shall not be less than four, unless otherwise approved by the Bridge Area Engineer.

3. Overhang. The minimum cap overhang shall be 450 mm.

19.1.7.2 Pile Design for Integral/Semi-Integral Abutments

The following criteria apply specifically to piles and loads at integral and semi-integral abutments:

1. Loads/Forces. For structures satisfying the requirements provided in Section 13.4.4, force effects in the abutment piles due to temperature, shrinkage, creep and horizontal earth pressures may be neglected.

   An alternative analysis must be used if the criteria in Section 13.4.4 are not met. The following steps should be considered in this analysis:

   a. The point of zero superstructure movement should be established by considering the elastic resistance of all substructures and bearing devices.

   b. The effects of creep, shrinkage and temperature should be considered.

c. Any movement at any point on the superstructure should be taken as being proportional to its distance to the point of zero deflection.

d. Lateral curvature of the superstructure may be neglected if it satisfies the provisions of Article 4.6.1.2 of the LRFD Specifications.

e. Vertical force effects in the abutment piles should be distributed linearly with load eccentricities properly accounted for.

f. Lateral soil resistance should be considered in establishing force effects and buckling resistance of piles.

g. Force effects should be combined in accordance with the provisions of Article 3.4.1 of the LRFD Specifications.

2. Pile Type. Only steel H-piles or steel pipe piles are permitted at integral abutments. For semi-integral abutments, steel H-piles, steel pipe piles or fluted steel piles are permitted. The orientation of steel H-piles (strong versus weak axis) is a design consideration, and it is preferable that all piles be oriented the same. All abutment piling shall be driven vertically and only one row of piling is permitted.

3. Pile Driving. Piles shall be driven a minimum of 3 m into natural ground. If piles cannot be driven to this depth due to an existing cohesive earth stratum, with a standard penetration resistance (N) exceeding 35 blows per 305 mm located with the 3 m interval below the bottom of the cap, the piles shall be placed in oversized, predrilled holes before driving. The diameter of the oversized holes should be 100 mm greater than the maximum cross sectional dimension of the pile. The holes shall be backfilled with uncrushed base course aggregate size 17 mm (pea gravel) following the pile driving operation.

   If piles cannot be driven a minimum of 3 m into natural ground due to a rock stratum, socket the piles into undersized holes drilled into the rock. The diameter of the undersized holes shall equal the inside diameter of the pipe pile, if pipe piles are used, or 25 mm less than the maximum pile dimension for steel H-piles. Socket the pile a minimum of 1 m into the rock formation; the pile should extend at least 3 m below the cap.

19.1.7.3 Pile Design for Free-Standing Abutments

The following criteria apply to piles at free-standing abutments:

1. Pile Spacing. At least two rows of piles or battered piles must be provided to provide the necessary longitudinal stiffness. The minimum pile spacing is 750 mm parallel to the centerline of the abutment.

2. Batter. Up to one-half of the piles may be battered to increase the overturning stability of the structure.

3. Movement. The effects of the movements due to overturning pressures or lateral pressures shall be investigated (e.g., ensure that the closing of joints does not occur).

19.1.8 Wingwalls

Reference: LRFD Article 11.6.1.4

Wingwalls shall be of sufficient length and depth to prevent the roadway embankment from encroaching onto the stream channel or the defined clear opening. Design the wingwall lengths to keep the embankment at least 300 mm below the beam seat or the top of the cap. Generally, the slope of the fill will not be steeper than 2:1 (H:V), and wingwall lengths will be established on this basis.

With respect to abutments, the following applies to wingwalls:

1. Pile Supported. If turnback wingwalls on rigid abutments have a total length of more than 3 m, auxiliary pile footings for wingwall support should be investigated. Pile-supported wings shall be avoided for integral backwalls.

2. Connections. In general, U-shaped wingwalls should not extend more than 3 m behind the rear face of the abutment. If wingwalls longer than 3 m are needed, force effects in the connection between the wingwall and abutment, and in the wingwall itself, shall be investigated and adequate reinforcing steel be provided. For rigid free-standing abutments, the forces are merely due to permanent loads and live-load surcharge. For flexible abutments, other transient loads must be considered in addition to the permanent loads.

3. Thickness. The minimum thickness of any wingwall with an abutment shall be 350 mm.

4. Design. Unattached wingwalls shall be designed as retaining walls.

5. Concrete. For wingwalls, use Class DD concrete.

19.1.9 Drainage

Provide positive drainage as needed in the embankment behind the abutment and wingwalls by using select backfill, weep holes, perforated drain pipe, a manufactured backwall drainage system or a combination of these options. Include provisions for select backfill in all abutment designs in accordance with the geotechnical recommendations in the Geotechnical Report.

Provide details of the selected drainage system on the bridge plans. Generally, the cost of furnishing and installing most systems can be absorbed in the cost of select backfill.

Static ground water levels should always be considered while evaluating an appropriate drainage system. Drainage systems should not be installed to allow pressurized backwater to saturate the abutment backfill during highwater events.

Generally, for relatively shallow girders supported on integral or semi-integral abutments with straight wings or turnback wings less than 3 m long, select backfill will be all that is needed to promote good drainage.

For bridges with taller abutment walls, girders deeper than 1.5 m or abutments with a total height of more than 2.5 m from the bottom of pile cap or footings to the top of the backwall should be given consideration for additional drainage features. If a drainage system is determined necessary, a perforated drainage pipe placed at the base of the abutment wall or footing is preferred. The pipe should be placed inside a free draining gravel media, wrapped in drainage fabric and sloped to drain to a point outside the abutment walls.

The other systems identified may be used to address site-specific needs with approval by the Bridge Area Engineer.

19.1.10 Joints

19.1.10.1 Construction Joints

To accommodate normal construction practices, the designer should indicate the following horizontal construction joints on the plans. MDT does not use shear keys for horizontal construction joints:

1. In semi-integral abutments, a horizontal construction joint shall be indicated between the bottom of slab fillet and the top of the backwall.

2. In integral abutments, in addition to the construction joint indicated between the bottom of slab fillet and the top of the backwall, a horizontal construction joint shall also be indicated at beam seat.

3. In free-standing abutments, a horizontal construction joint shall be indicated on the drawings between the top of the cap or footing and the bottom of the backwall. Some expansion joint types may require another construction joint at the bottom of the paving notch.

4. In turnback wings, a horizontal construction joint shall be indicated at an elevation the same as the top of the cap.

Planned vertical construction joints are normally associated with phase construction issues or perhaps close proximity to an existing structure. Provision needs to be made for splicing or mechanical rebar couplers on horizontal reinforcing steel. Vertical reinforcing steel should be at least 75 mm from the construction joint.

19.1.11 Concrete

Use Class DD concrete for all substructure components.

19.2 INTERMEDIATE SUPPORTS

Reference: LRFD Article 11.7

19.2.1 Types

MDT uses four basic types of intermediate supports for bridges, which are discussed in the following sections. Also, see Section 13.4.7 for more information.

19.2.1.1 Pipe Pile Bents

Under the right conditions, pipe pile bents may provide the most economical substructure. Do not use this type of bent in the presence of large horizontal forces. Note that debris accumulation can increase stream and ice forces significantly.

19.2.1.2 Piers

MDT uses two types of piers:

1. Single Wall. This is a wall set on a spread footing or a pile cap with multiple rows of piles.

2. Hammerhead. For larger structural heights and pier widths, a hammerhead pier (either with rectangular or rounded stem) is often more suitable. The strut-and-tie model of LRFD Article 5.6.3 should be considered where the length of the cantilever is less than twice the depth of the cantilever.

19.2.1.3 Multi-Column Bents

Concrete frame bents may be used to support a variety of superstructures. The columns of the bent may be either circular or rectangular in cross section. The columns may be directly supported by the footing or by a partial height wall. If the columns rest directly on the footing, the footing shall be designed as a two-way slab.

19.2.1.4 Single-Column Piers

The round column is commonly used because of its ease of design, its concrete confinement for seismic and its multi-directional flow characteristics.

19.2.2 General Design Considerations

In general, the following design criteria apply to intermediate supports, where applicable:

1. Piers in Waterways. Wall piers should have a solid wall to an elevation of 300 mm above the Q_{100} high-water level. Depending on aesthetics and economics, the remainder of the wall may be either solid or multiple columns. The dimensions of the wall may be reduced by providing cantilevers to form a hammerhead pier. River piers shall have ice protection. The steel protectors may be in the form of angles, casings or plates. The nose plates or angles shall extend from the channel bottom to 300 mm above the Q_{100} high-water elevation on the upstream end of the pier only.

2. Footings. Bents founded on spread footings have typically been designed with separate footings under each column. Existing analytical techniques provide tools for the analysis of a common footing for all columns, and this configuration may result in a more economical footing.

3. Highway Bridge Over Railroad. See Chapter Twenty-one for more information.

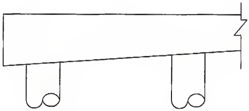
4. Column Reinforcement. Column vertical bars shall extend into the cap beam to within 50 mm of the top reinforcement. The vertical column bars must be fully developed when they exit the cap beam and the spread footing or pile cap.

5. Size. For spread footings of piers or bents in rivers, the least ratio of footing width to bent or pier height shall be 1:4. For pile footings of piers or bents in rivers, the least ratio of

pile-group width to bent or pier height shall be 1:4. For dry-land structures, the least ratio of spread-footing or pile-group width to bent or pier height shall be 1:5. Columns are typically rectangular, square or round, with a minimum diameter or thickness of 600 mm. Diameter increments shall be in multiples of 150 mm. Solid pier walls shall have a minimum thickness of 600 mm. If conditions warrant, caps up to 300 mm wider than the thickness or diameter of columns may be used. Caps shall be at least 80 mm wider than the thickness or diameter of the columns.

6. Cap Extension. The width of caps shall project beyond the sides of columns. The added width of the cap shall be a minimum of 40 mm on the outside of the column. This width will reduce the reinforcement interference between the column and cap. The cap should preferably have cantilevered ends to balance positive and negative moments in the cap.

7. Step Caps. Where one end of the cap is on a considerably different elevation than the other, the difference shall be accommodated by increasing the column heights as shown below:



The bottom of the cap shall be sloped at the same rate as the cross slope of the top of the bridge deck.

8. Epoxy-Coated Steel Under Expansion Joints. All reinforcing steel in cap beams at intermediate piers where an expansion joint is located directly over the cap shall be epoxy coated. Note that this does not apply to all piers. It applies only to those substructures which support the ends of two superstructure units with an expansion joint located directly over the cap. Because most structures are single continuous units, this type of substructure is relatively uncommon and will generally occur only on long structures with multiple continuous units.

9. Concrete. For intermediate supports, use Class DD concrete.

10. Steel Splices. If a pier is less than 3 m in height, do not splice the steel extending out of the footing. For small columns with a high percentage of vertical steel and for columns in seismically active regions, mechanical connectors should be used for splicing the vertical steel. No splices may be located within the plastic regions of the column and, where used elsewhere, they should be staggered.

11. Compressive Steel. Compressive steel tends to buckle when the cover is gone or when the concrete around the steel is weakened by compression. The criteria in the LRFD Specifications, Article 5.7.4.6 or 5.10.11, for ties and spirals, should be rigidly adhered to.

12. Minimum Edge Distance for Anchor Bolts. The edge distance from the center of the anchor bolt to the edge of the cap shall be 250 mm.

19.2.3 Specific Design Criteria

This Section presents design criteria which applies to the specific type of intermediate support.

19.2.3.1 Pipe Pile Bents

The following applies to the design of pipe pile bents:

1. Limitations. This type of support has a relatively low resistance to longitudinal forces. This support should also not be used if the stream carries large debris or heavy ice flow. Scour should be considered in establishing design pile lengths and for the structural design of the pile.

2. Cap Beam. Pile bents always need a cap beam for structural soundness, which may be an integral part of the superstructure.

3. Loads. Because the piles are relatively flexible compared to the abutments, the force effects induced in the piles by lateral displacement is small. Where practical, one pile should be placed beneath each girder line. The vertical load carried by the piles shall be the girder reaction and the appropriate portion of the pile cap dead load. Assuming the bent acts as a rigid frame in a direction parallel to the bent, force effects due to lateral displacement and lateral loads may be uniformly distributed among the piles.

19.2.3.2 Hammerhead Piers

The following applies to the design of hammerhead piers:

1. The bottom of a hammerhead cap should preferably be a minimum of 2 m above the finished ground line on stream crossings to help prevent debris accumulation.

2. The design of the cantilever is affected by the cantilever depth versus length geometry. The strut-and-tie model of LRFD Article 5.6.3 should be considered where the length of the cantilever is less than twice the depth of the cantilever. Otherwise, the sectional models for moment and shear are appropriate.

3. Non-contact splices should not be used at the connection of the bottom of the cap beam to the column.

4. Architectural treatments should be discussed at the Design Parameters Meeting.

19.2.4 Pier and Bent in a Sloped Embankment

For piers or bents located in the sloped portion of an embankment, the earth pressure against the back of the footing and column shall be increased 100% to include the effect of adjacent embankment. The effect of the embankment in front of the pier or bent shall be neglected. Piers and bents located in the embankment shall be investigated for stability not considering the superstructure loads.

19.2.5 Dynamic Load Allowance (IM) for Piers and Bents

Reference: LRFD Article 3.6.2.1

Dynamic Load Allowance (IM), traditionally called impact, shall be included in the design of piers and bent columns, but shall not be applied to the design of their footings.

19.3 BEARINGS

19.3.1 General

Reference: LRFD Articles 14.4 and 14.6

Bearings ensure the functionality of a bridge by allowing translation and rotation to occur while supporting the vertical loads. For most normal applications, MDT uses two bearing types. They are steel rocker plates and elastomeric bearing pads.

Steel rocker plates are commonly used for fixed bearings on both prestressed concrete and steel girder bridges where no longitudinal movement of the bearing is required. Standard steel bearing details for each standard prestressed girder are shown in the standard girder drawings.

Elastomeric bearings are typically used for steel girder bridges or for special conditions on prestressed concrete girder bridges. Elastomeric bearings need to be designed for each location within a structure and can be designed as either fixed bearings or expansion bearings to provide for longitudinal movement at the beam end. MDT's current design practice generally results in the use of steel reinforced elastomeric bearings. Although plain elastomeric bearings may be considered for special situations, steel reinforced bearings are more common.

Both steel rocker plates and elastomeric bearings provide for girder end rotations about an axis perpendicular to the girder centerline. When selecting and designing bearings for a bridge, the designer must consider the type of superstructure, span lengths, span arrangement, substructure and foundation conditions. Bearings will be designed to accommodate needed girder end rotations and movements in the longitudinal direction. MDT's typical bearing designs do not account for rotation or translation in the transverse direction.

The following will apply:

1. Movements. Consideration of movement is important for bearing design. Movements include both translations and rotations. The sources of movement include bridge skew and horizontal curvature effects, initial camber or curvature, construction loads, misalignment or construction tolerances, settlement of supports, thermal effects, creep, shrinkage and traffic loading. Bearing pads on skewed structures should be oriented parallel to the principal rotation axis. Where insufficient seat width exists, the bearing pads may be oriented normal to the support.

2. Effect of Bridge Skew and Horizontal Curvature. Skewed bridges move both longitudinally and transversely. The transverse movement becomes significant on bridges with skew angles greater than 20 degrees that have bearings not oriented parallel to the movement of the structure.

   Curved bridges move both radially and tangentially. These complex movements are predominant in curved bridges with small radii and with expansion lengths that are longer than 60 m.

   MDT does not typically consider the effects of skew. For large bridges with unusual geometry, these movements may need consideration.

   The effect of curvature is normally addressed in expansion bearings by orienting the slots in the sole plates parallel to the span chord.

3. Effect of Camber and Construction Procedures. The initial camber of bridge girders and out-of-level support surfaces induce bearing rotation. Initial camber may cause a larger initial rotation on the bearing, but this rotation may grow smaller as the construction of the bridge progresses. Rotation due to camber and the initial construction tolerances are sometimes the largest component of the total bearing rotation. Due to the short duration of the

initial rotation from application of the dead load of the slab, it is MDT's design practice to not account for dead load rotations in the design of the bearings and to assume that the pads are equally stressed across their full width after application of full dead load. Pads will be designed for rotations of 0.005 radians to account for construction irregularities. In addition, include live load rotation in the pad design. Longitudinal girder slope is accounted for by beveling the sole plate for slopes greater than 2% or where the thickness of the sole plate varies more than 2 mm across the width of the plate. The curved surfaces on the steel rocker plate bearings will typically account for dead load and live load rotations without additional consideration.

4. Thermal Effects. Thermal translation, ΔL, is estimated by:

   $\Delta L = \alpha \, (L_E) \, (\Delta T)$

   where L_E is the expansion length and α is the coefficient of thermal expansion, use 10.8×10^{-6} / °C for normal density concrete and 11.7×10^{-6} / °C for steel, and ΔT is the change in the average bridge temperature. A change in the bridge temperature causes thermal translation. Maximum and minimum bridge temperatures for bearing design are defined the same as for expansion joint design in bridge decks (see Section 15.3.7) as –40° C to 45° C. The change in bridge temperature (ΔT) between the installation temperature and the design extreme temperatures is used to compute the positive and negative movements. To reduce extreme movements in one direction or the other, it is desirable to lock down the fixed bearings near the mean temperature. To reduce thermal stresses in the bridge or bearing movements, it may be desirable in some situations to specify welding the bearings at a temperature close to mean. It should be further noted that a given temperature change causes thermal movement in all directions of the bridge; however, this is rarely accounted for in design.

5. Loads and Restraint. Restraint forces occur when any part of a movement is prevented. Forces due to direct loads include the dead load of the bridge and loads due to traffic, earthquakes, water and wind. Temporary loads due to construction equipment and staging also occur. The majority of the direct design loads are reactions of the bridge superstructure on the bearing, and they can be estimated from the structural analysis. The applicable AASHTO load combinations specified in LRFD Article 3.4.1 must be considered.

6. Serviceability, Maintenance and Protection Requirements. Bearings under deck joints collect large amounts of dirt and moisture and promote problems of corrosion and deterioration. As a result, these bearings should be designed and installed to have the maximum possible protection against the environment and to allow easy access for inspection.

   The service demands on bridge bearings are very severe and result in a service life that is typically shorter than that of other bridge elements. Therefore, thought should be given in the design process to bearing maintenance and replacement. The primary requirements are to allow space suitable for lifting jacks during the original design and to employ details that permit quick removal and replacement of the bearing.

7. Clear Distance. The minimum clear distance between the bottom shoe of a steel bearing and the edge of the bearing seat or cap shall be 75 mm. For elastomeric pads resting directly on the concrete bridge seat, the minimum edge distance shall be 75 mm as well, except under deck expansion joints where 150 mm is required. The required distance from the center of anchor bolts to the nearest edge of concrete is 250 mm. Seismic support lengths must be checked and Code requirements met.

8. Bearing Selection. Bearing selection is influenced by many factors such as loads, geometry, maintenance, available clearance, displacement, rotation, deflection, availability, policy, designer preference, construction tolerances and cost.

   In general, vertical displacements are prevented, rotations are allowed to occur as freely as possible, and horizontal displacements may be either accommodated or prevented. The loads should be distributed among the bearings in accordance with the superstructure analysis.

   Unless conditions dictate otherwise, conventional steel radius plate bearings should be used for fixed shoes of prestressed girder bridges and small steel girder structures. All expansion bearings of both steel and prestressed girder bridges and fixed bearings of larger steel bridges will be designed using elastomeric bearings. Plain elastomeric bearings will accommodate small amounts of movement; however, when the practical limits of the plain bearing pads are exceeded, the designer must consider using Polytetrafluorethylene (PTFE) sliding bearings, commonly referred to as Teflon or TFE bearings, in conjunction with a stainless steel sliding surface and a steel-reinforced elastomeric bearing pad. See Figure 19.3A for a general summary of expansion bearing capabilities. The values shown in the table are for general guidance only. For large or unusual structures not commonly constructed in Montana, more elaborate bearing systems may be required.

   The final step in the selection process consists of completing a design of the bearing in accordance with the LRFD Specifications. The resulting design will provide the geometry and other pertinent specifications for the bearing.

   On structure widenings, the designer is cautioned against mismatching bearing types. Yielding type bearings, such as elastomeric, should not be used in conjunction with non-yielding type bearings.

   Girder bridges without integral abutments must have at least one fixed bearing line. If integral abutments meeting the empirical design limits outlined in Chapter 19 are used, interior fixed bearings are not required.

9. Anchor Bolts. Use swedged anchor bolts to connect all steel and elastomeric bearing assemblies to the concrete beam seat. Bolts will be sized to accommodate anticipated longitudinal and transverse design forces. Where anchor bolts lie within the confines of the backwall on semi-integral abutments, use smooth dowel rods with expansion caps to allow for future grade adjustments.

19.3.2 Fixed Steel Bearings

19.3.2.1 General

The top plate of steel bearings shall be at least as wide as the bottom girder flange plus sufficient added width to accommodate the anchor bolts and nuts.

When the flexibility of tall, slender piers is sufficient to absorb the horizontal movement at the bearings due to temperature change without developing undue force in the superstructure, bearings or pier, then two or more piers may be fixed to distribute the longitudinal force among the piers.

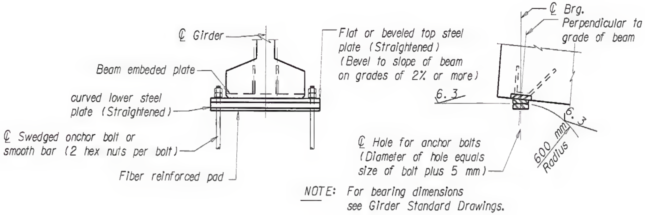
19.3.2.2 Design

Figure 19.3B illustrates representations of the steel rock plate bearings used by MDT.

Bearing Type	Load		Translation		Rotation	Costs	
	Min. (kN)	Max. (kN)	Min. (mm)	Max. (mm)	Limit (Rad.)	Initial	Maintenance
Elastomeric Pads							
Plain (PEP)	0	450	0	15	0.01	Low	Low
Cotton Duck-Reinforced(CDP)	0	1400	0	5	0.003	Low	Low
Fiberglass-Reinforced(FGP)	0	600	0	25	0.015	Low	Low
Steel-Reinforced Elastomeric Bearing	225	3500	0	100	0.04	Low	Low
Flat PTFE Slider (Polytetrafluorethylene)	0	>10,000	25	>100	0	Low	Moderate
Curved Sliding Cylindrical	0	7000	0	0	> 0.04	Moderate	Moderate
Pot Bearing	1200	10,000	0	0	0.02	Moderate	High
Curved PTFE	1200	7000	0	0	> 0.04	High	Moderate

SUMMARY OF EXPANSION BEARING CAPABILITIES

Figure 19.3A



STEEL ROCKER PLATE SHOE DETAILS

Figure 19.3B

The fixed shoe details are representative of typical steel rocker plate bearings discussed previously in this Chapter. Standard fixed shoe details are included on each standard prestressed girder drawing and need not be designed or covered elsewhere in the contract plans and specifications. Fixed shoe details for steel bridges or other non-standard applications will need to be designed and shown on the plans. Design requirements are simply to size the bearing such that concrete and steel stresses remain within an acceptable range throughout the controlling service and extreme event load conditions. Typical design checks would be for compression of the concrete under the bearing plate and for bearing plate bending about the bottom flange of the beam. Bearing anchor bolts will be designed to resist the resulting stresses from the combined transverse and longitudinal forces applied at the bearings.

19.3.3 Steel- Reinforced Elastomeric Bearings

Reference: LRFD Articles 14.7.5 and 14.7.6

The behavior of steel-reinforced elastomeric bearings is influenced by the shape factor (S) where:

$$S = \frac{\text{Plan Area}}{\text{Area of Perimeter Free to Bulge}}$$

It is usually desirable to orient elastomeric bearings so that the long side is parallel to the principal axis of rotation, because this facilitates the accommodation of rotation. If holes are placed in a steel-reinforced bearing, the steel reinforcement thickness should be increased in accordance with LRFD Article 14.7.5.3.7.

Steel-reinforced elastomeric bearings have many desirable attributes. They are usually a low-cost option, and they require minimal maintenance. Further, these components are relatively forgiving if subjected to loads, movements or rotations that are slightly larger than those considered in their design. This is not to encourage the engineer to underdesign elastomeric bearings, but it simply notes that extreme events, which have a low probability of occurrence, will have far less serious consequences with these elastomeric components than with other bearing systems.

19.3.3.1 Elastomer

Reference: LRFD Articles 14.7.5.2 and 14.7.6.2

Both natural rubber and neoprene are used in the construction of bridge bearings. The differences between the two are usually not very significant. Neoprene has greater resistance than natural rubber to ozone and a wide range of chemicals, and so it is more suitable for some harsh chemical environments. However, natural rubber generally stiffens less than neoprene at low temperatures.

All elastomers are visco-elastic, nonlinear materials and, therefore, their properties vary with strain level, rate of loading and temperature. Bearing manufacturers evaluate the materials on the basis of Shore A Durometer hardness, but this parameter is not a good indicator of the shear modulus, G. A Shore A Durometer hardness of 50 to 60 will be used in Montana, and this leads to shear modulus values in the range of 0.78 to 1.14 (use least favorable value for design) MPa @23°C. The shear stiffness of the bearing is its most important property because it affects the forces transmitted between the superstructure and substructure.

Elastomers are flexible under shear and uniaxial deformation, but they are very stiff against volume changes. This feature makes possible the design of a bearing that is stiff in compression but flexible in shear.

Elastomers stiffen at low temperatures. The low-temperature stiffening effect is very sensitive to the elastomer compound and the increase in shear resistance can be controlled by selection of an elastomer compound that is appropriate for the climatic conditions. For Montana, the minimum low-temperature elastomer shall be Grade 4, unless Special Provisions are used, in which case Grade 3 is acceptable. The designer shall indicate the elastomer grade in the contract documents.

19.3.3.2 Behavior of Steel-Reinforced Elastomeric Bearing Pads

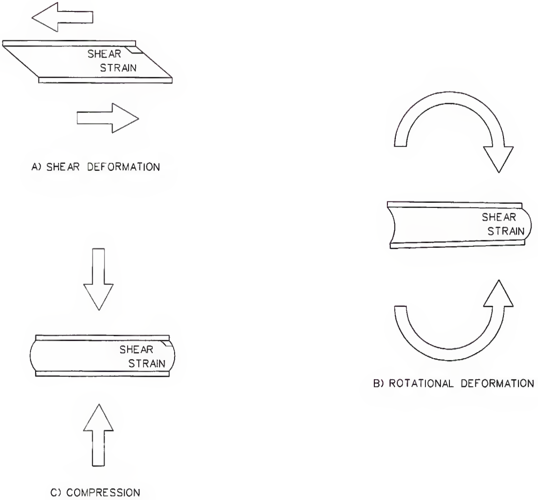
Steel-reinforced elastomeric bearings are often categorized with elastomeric bearing pads, but the steel reinforcement makes their behavior quite different. Steel-reinforced elastomeric bearings have uniformly spaced layers of steel and elastomer. The bearing accommodates translation and rotation by deformation of the elastomer. The elastomer is flexible under shear stress but stiff against volumetric changes. Under uniaxial compression, the flexible elastomer would shorten significantly and sustain large increases in its plan dimension, but the stiff steel layers restrain this lateral expansion. This restraint induces a bulging pattern as shown in Figure 19.3C and provides a large increase in stiffness under compressive load. This permits a steel-reinforced elastomeric bearing to support relatively large compressive loads while accommodating large translations and rotations.

The design of a steel-reinforced elastomeric bearing pad requires an appropriate balance of compressive, shear and rotational stiffnesses. The shape factor affects the compressive and rotation stiffness, but it has no impact on the translational stiffness or deformation capacity.

A bearing pad must be designed to control the stress in the steel reinforcement and the strain in the elastomer. This is done by controlling the elastomer layer thickness and the shape factor of the bearing. Fatigue, stability, delamination, yield and rupture of the steel reinforcement, stiffness of the elastomer, and geometric constraints must be satisfied.

Large rotations and translations require taller bearings. Translations and rotations may occur about the longitudinal or transverse axis of a steel-reinforced elastomeric bearing.

Steel-reinforced elastomeric bearings become large if they are designed for loads greater than about 3000 kN. Uniform heating and curing during vulcanization of such a large mass of elastomer becomes difficult, because elastomers are poor heat conductors. Manufacturing constraints thus impose a practical upper limit on the size of most steel-reinforced elastomeric bearings. If the design loads exceed 3000 kN, the designer should check with the manufacturer for availability.



STRAINS IN A STEEL REINFORCED ELASTOMERIC BEARING

Figure 19.3C

19.3.4 Design of Steel-Reinforced Elastomeric Bearing Pads

Reference: LRFD Articles 14.7.5 and 14.7.6

Steel-reinforced elastomeric bearings may be designed using either of two methods, commonly referred to as Method A and Method B. The Method A procedure, which is typically used in Montana, found in the LRFD Specifications, Article 14.7.6 shall be used for conventional elastomeric bearings. The Method B procedure found in the LRFD Specifications, Article 14.7.5 shall be used for high-capacity bearings, which are not typically used by MDT.

Design criteria for both Methods are based upon satisfying fatigue, stability, delamination, steel-reinforcement yield/rupture, and elastomer stiffness requirements. The design of a steel-reinforced elastomeric bearing requires an appropriate balance of compressive, shear and rotational stiffnesses. The shape factor, as defined by the steel shim spacing, significantly affects the compressive and rotational stiffness of the bearing. However, it has no impact on the translational stiffness of the bearing or its translational deformation capacity.

The minimum elastomeric bearing length or width shall be 150 mm. All pads shall be 50 to 60 durometer hardness. For overall bearing heights less than 90 mm, a minimum of 3 mm of side clearance shall be provided beyond the edges of the steel shims. For overall heights over 90 mm, a minimum of 6 mm of side clearance shall be provided. The top and bottom cover layers shall be no more than 70 percent of the thickness of the interior layers.

In determining bearing pad thicknesses, it should be assumed that slippage will not occur. The total elastomer thickness shall be no less than twice the maximum longitudinal or transverse deflection. If the factored shear force sustained by the deformed pad at the strength limit state exceeds one-fifth of the minimum vertical force due to permanent loads, the bearing shall be secured against horizontal movement.

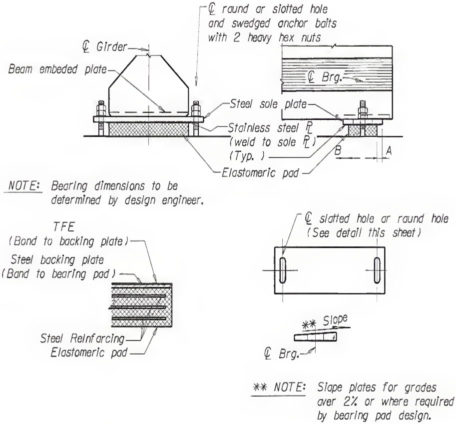
Figure 19.3D illustrates representations of the elastomeric bearings used by MDT.

Plain or reinforced elastomeric bearings, whether fixed or expansion, are to be custom designed for each required location within the structure. It is the project design engineer's responsibility to size the pads, plates and anchorage and to provide the design information to the detailer to be placed on the plans. General required plan information is shown in Figure 19.3D. Modify this information as needed for specific situations. Include the size, thickness and layering information of the pad, the size, thickness hole, dimensions and beveling of the sole plate and the anchor bolt size. Also required is a Table of Expansion Shoe Dimensions with the shoe adjustment per 1 degree of temperature change. The temperature range, total design movement and the bearing pad design load must be documented in the plans.

Design the pads using Method A, Article 14.7.6 of the LRFD Specifications or MDT's internal design software documented in Chapter 25 of this **Manual**. For normal situations, calculations for design movements can be limited strictly to temperature change. As a general rule, shrinkage and creep calculations need not be included in the design movement for bearing pads. For fixed shoes, the holes in the plate will typically be 5 mm larger than the bolt diameter. For expansion bearings slotted holes should be 5 mm wider than the bolt and sized in length to accommodate the full design movement plus two times the bolt diameter. Sliding surfaces will be TFE on stainless steel. TFE is bonded to the elastomeric pad and the stainless steel is welded to the steel sole plate. Sole plate dimensions are to be larger than the pad and TFE so that the TFE is fully protected from dirt and debris during the full range of shoe movement.

19.3.5 Seismic Design

This Section discusses seismic design for bearing assemblies.



ELASTOMERIC SHOE DETAIL

Figure 19.3D

19.3.5.1 Application

All bridges shall be designed in accordance with the LRFD Specifications. Most of Montana is classified by AASHTO as being in Seismic Performance Zone 1. The Missoula and Butte Districts, however, are characterized as being of high seismic risk with acceleration coefficients high enough to significantly affect the bridge designs.

19.3.5.2 Seismic Performance Zone 1 Criteria

Reference: LRFD Articles 3.10.9 and 4.7.4.4

All bridges shall comply with the following LRFD Specifications criteria for Zone 1:

1. Minimum Support Length. Adequate support length is probably the most important contributor to satisfactory performance of a bridge during a seismic event. The support length required by Article 4.7.4.4 of the LFRD Specifications shall be provided at the expansion ends of all structures unless longitudinal restrainers are provided.

2. Minimum Bearing Force Demands. The connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to 0.20 times the tributary dead load force in the restrained directions. No additional adjustment factors, loading cases or friction forces shall be applied to increase or decrease this minimum horizontal seismic force. This force shall extend into the

substructure design as an extreme event load case.

Fixed bearings, such as steel shoes, shall be attached to the pier cap with anchor bolts. Some examples of acceptable means of restraint at semi-fixed or expansion bearings in Zone 1 include concrete shear keys, beams resting in concrete channels and steel side retainers bolted to the cap.

In designing the bearing connections for Zones 2, 3 and 4, the actual calculated seismic design forces, as adjusted by Article 3.10.7 of the LRFD Specifications, shall be used. The longitudinal seismic forces at expansion bearings may be resisted either by using seismic restraining devices (positive horizontal linkage), or they may be transferred to the bearing connections at the nearest fixed pier. Positive linkage shall be provided by ties, cables, dampers or other equivalent mechanism. Friction shall not be considered a positive linkage.

See Article 3.10.9.6 of the LRFD Specifications to determine if hold-down devices are required.

19.3.5.3 Connections for Fixed Steel Shoes

The connection between a fixed steel shoe and the pier cap shall be made with anchor bolts. The anchor bolts, the pintles and the high-strength bolts in the top shoe shall be verified that their ultimate shear resistance is adequate to resist the calculated seismic forces. See Article 6.13.2.7 of the LRFD Specifications for determining the nominal shear resistance of anchor bolts and pintles.

The masonry anchor bolts shall extend into the concrete a minimum of 380 mm, and anchor bolts used in seismic performance Zone 2, 3 and 4 shall meet the requirements of Article 14.8.3 of the LRFD Specifications.

Anchor bolts should be located beyond the limits of the bottom flange and avoid conflict with interior diaphragms. Provide adequate clearance for installation of the nuts. The grade of structural steel used for the anchor bolts or pintles shall be clearly indicated in the plans.

19.3.5.4 Connections for Elastomeric Bearings and PTFE/Elastomeric Bearings

All elastomeric PTFE/elastomeric bearings shall be provided with adequate seismic-resistant anchorage to resist the transverse horizontal forces in excess of those accommodated by shear in the bearing. The restraint may be provided by one of the following methods:

1. steel side retainers with anchor bolts;

2. concrete shear keys placed in the top of the pier cap, or channel slots formed into the top of cap at the abutments (see Section 19.3.5.5); or

3. concrete channels formed in the top of abutment caps or expansion pier caps.

Steel side retainers and the anchor bolts shall be designed to resist the minimum transverse seismic force for the zone in which the bridge is located. The number of side retainers shall be as required to resist the seismic forces and shall be placed symmetrically with respect to the cross section of the bridge. Many times, side retainers will be required on each side of the girder flange of each beam line. The strength of the beams and diaphragms shall be sufficient to transmit the seismic forces from the superstructure to the bearings.

Concrete channels or shear blocks formed around each beam in the top of abutment caps or expansion pier caps represent an acceptable alternative to steel side retainers. The top of the top shoe plate shall be set a minimum of 50 mm below the top of the concrete channel. The minimum depth of the channel shall be 150 mm. The horizontal clearance from the side of the top shoe or edge of beam to the side wall of the channel shall be 15 mm or less. Adequately reinforce all shear blocks and channels to resist the applied loads.

Integral abutments are a very effective way of accommodating the horizontal seismic forces of Zones 1 and 2. An integrally designed abutment will inherently resist the transverse seismic forces. Minimum support length requirements need not be checked for this type of substructure, provided that the beams are adequately connected to the wall. See Section 19.1.5 for integral abutment requirements.

19.3.5.5 Seismic Isolation Bearings

The use of seismic isolation bearings should be considered for seismic retrofit of continuous steel bridges in Seismic Zones 2, 3 and 4. MDT's experience indicates that the savings in substructure rehabilitation cost, resulting from an isolation bearing design, roughly offsets the substantial cost of the isolation bearings. The use of seismic isolation bearings should be based on performing a cost analysis comparing other alternatives, such as elastomeric bearings with suitable retainers or longitudinal restraining devices. The use of seismic isolation bearings in Seismic Zone 1 is not cost effective.

The minimum bearing support length requirements of the LRFD Specifications for seismic design shall be satisfied at the expansion ends of bridges with seismic isolation bearings. The minimum bearing force demands should be assumed to be the actual calculated seismic forces.

Seismic isolation bearings significantly reduce the seismic forces on the substructure, possibly to the point where a non-seismic load case may control the pier design. This, however, does not relieve the designer of the need to provide pile anchorage, confinement steel in plastic hinge regions and proper location of lap splices. The design of seismic isolation bearings shall be in accordance with the AASHTO **Guide Specifications for Seismic Isolation Design,** 1999. The LRFD Specifications requires that all bearing systems shall be tested under both static and cyclic loading prior to acceptance. The designer shall prepare a Special Provision, which includes the testing requirements that will be the responsibility of the bearing supplier.

Table of Contents

Chapter Twenty
FOUNDATIONS

A critical consideration for the satisfactory performance of any structure is the proper selection and design of foundations that will provide adequate resistance, tolerable lateral and vertical movements and aesthetic compatibility. This Chapter discusses MDT-specific criteria for the design of structural foundations for spread footings, driven piles and drilled shafts.

20.1 GENERAL

Chapter Twenty is based upon the LRFD design approach. The following summarizes the concepts in the LRFD Specifications.

20.1.1 LRFD Specifications

Considering basic design principles for foundations, the **LRFD Bridge Design Specifications** has implemented a major change compared to those principles in the former AASHTO **Standard Specifications for Highway Bridges**. The LRFD Specifications makes a clear distinction between the strength of the native materials (soils and rocks) supporting the bridge and the strength of the structural components transmitting force effects to these materials. The distinction is emphasized by treating the former in Section 10 "Foundations" and the latter in Section 11 "Abutments, Piers and Walls." It is necessitated by the substantial difference in the reliability of native materials and man-made structures. The foundation provisions of the LRFD Specifications are essentially strength design provisions with a primary objective to ensure equal, or close to equal, safety levels in all similar components against structural failure. The target safety levels for each type of foundation are chosen to achieve a level of safety comparable with that inherent in those foundations designed with the former Standard Specifications This approach differs from that for superstructures, where a common safety level has been selected for all superstructure types.

Historically, the primary cause of bridge collapse has been the washout of native materials. Other substructure/foundation failures, other than those precipitated by vessel or vehicular collision, are virtually non-existent. Accordingly, the LRFD Specifications introduced a variety of strict provisions in scour protection, which may result in deeper foundations.

To ensure maximum efficiency, the LRFD Specifications requires that components of the substructure foundation be analyzed and proportioned no differently from those of the superstructure. In practical terms, this means that force effects in the substructure and between the substructure and foundation are determined by analysis, as appropriate, and factored according to Section 3 of the LRFD Specifications. Loads generated by earth pressures can be determined with assistance from Section 11. Then, the nominal and factored resistance of the substructure is computed according to Section 10. The geotechnical resistance factors provided in Tables 10.5.5-1, -2 and -3 of the LRFD Specifications are approximately half of those provided for structural components. This is the justification for the new design philosophy, which permits the designer to tailor the level of design sophistication to the size, importance and appearance of the bridge.

Sections 10 and 11 of the LRFD Specifications are largely based on NCHRP Report 343 **Manuals for the Design of Bridge Foundations**.

20.1.2 Required Information

Prior to the design of the foundation, the designer must have a knowledge of the environmental, climatic and loading conditions expected during the life of the proposed unit. The primary function of the foundation is either 1) to spread concentrated loads over a sufficient area to provide adequate bearing capacity and limitation of movement, or 2) to transfer loads from unsuitable foundation strata to suitable strata. Therefore, a knowledge of the subsurface soil conditions, location and quality of rock, ground water conditions, and scour and frost effects is necessary.

For the Geotechnical Section to perform its analysis, the Section must know the magnitude and types of loads that require support. The accepted practice is for the Bridge Bureau to report the loads and reactions from the bridge superstructure at the ground line. The Geotechnical staff will analyze only the portion of the shaft or pile below ground. The Geotechnical Engineer will forward the L-Pile files to the bridge designer for the Bridge Bureau's use in designing portions of the bridge substructure that are above ground line, using the same program as Geotechnical. The bridge designer uses the L-Pile files to examine pile top deflections and the behavior of the substructure under different loading combinations. If any modifications are necessary, such as embedment depth or pile diameter, then the geotechnical designer will perform a new foundation analysis.

Conceptual axial loads need to be provided with the Core Request for the Geotechnical Section to determine proper methods for core sampling. Service loads should be reported. After sampling and analysis, Geotechnical will deliver core logs to the Bridge Bureau and a recommendation for foundation type.

Geotechnical will analyze the foundation with these loads. The Section will send a preliminary report to the Bridge Bureau plus an L-Pile file containing the soils and foundation information. The bridge designer may use the file to analyze the substructure elements above ground.

If the final axial design loads are within 10% of the preliminary loads used for the foundation analysis, no further foundation analysis is necessary. If the final design loads are outside this envelope, discuss with Geotechnical the necessity and advisability for further foundation analysis. Figure 20.1A shows the format and information provided in a typical core request.

20.1.3 Selection of Foundation Type

Section 13.4.8 discusses those types of foundations used by MDT and the general criteria which influence the selection of a foundation type.

Typically, the selection of a foundation type is based on the foundation investigation and recommendations by the Geotechnical Section. The selection is made by examining the test boring data, the existing ground lines, whether or not the proposed roadway is below, at or above the existing ground line, and hydraulic considerations such as scour depth or the desirability of a multidirectional pier.

20.1.4 Location of Bottom of Foundation

Figure 20.1B provides guidance on basic design criteria for the elevations of footings and pile tips.

20.1.5 Foundation Approval

The Bridge Bureau selects a structural foundation type based on boring information and the Geotechnical Section's recommended foundation type. The information presented for consideration of foundation type should include the following:

1. logs of subsurface investigation;
2. plan and elevation showing proposed foundations with applicable test borings plotted at the proper location and elevation;

3. allowable foundation pressure or type, size and maximum allowable load of piles. When piles are proposed, the estimated pile tip elevation at each foundation must be shown; and

4. finished ground elevation at the face of substructure.

MONTANA DEPARTMENT OF TRANSPORTATION
Helena, Montana 59620

MEMORANDUM

TO: Kent Barnes, P.E.
Materials Engineer

FROM: Joseph P. Kolman, P.E.
Bridge Engineer

DATE: August 29, 2001

SUBJECT: F-NH 1-3(20)247
Cut Bank West
CN 1310

Please furnish Borings and Foundation Recommendations for the proposed replacement bridge over Cut Bank Creek on US 2 just west of the community of Cut Bank. The new bridge will be constructed on a new alignment located just downstream from the existing bridge. The Alignment Review activity is complete and the alignment approved.

The attached bridge layout represents our proposed structure concept for this site. The preliminary estimated DL + LL reactions at the ground line are indicated below. Lateral loads will be furnished later as the design progresses.

End Bents: 634 kn/pile (estimate based on 5 beam lines and 2 piles/beam at each end)
Int. Piers: 9127 kn/drilled shaft (we plan to use 1 drilled shaft/pier)

Project Location:
Township/Range/Section
Glacier County:
T33N, R6W, Section 11 (See attached Map)

JPK:rwm:1310core-req

Attachments
CC: M.P. Johnson – Great Falls
R.E. Williams – Road Design
K.M. Barnes, w/1 attached
J.J. Moran, Geotechnical Section, w/1 attach
A.Kornec, Core Drill Section, w/1 attach
G.J. Stockstad, Environmental Services, w/1 attach
R.W. Modrow, Bridge Bureau, w/1 attach
Bridge File, w/1 attach

Figure 20.1A

- *Intermediate Bent*
  - *Pile Bent*
    - *Friction Piles*
      - *Attain design capacity below Q_{100} total scour depth.*
    - *End Bearing Piles*
      - *Minimum pile tip elevation 6.0 m below Q_{100} total scour depth.*
  - *Spread Footing*
    - *On Rock*
      - *Place bottom of footing 0.90 m (min.) below surface of scour-resistant rock. Geotechnical Section will verify depth during construction. Place top of footing below the rock surface (min.).*
    - *On Piles*
      - *Place top of footing below Q_{100} contraction scour elevation, bottom of footing 1.80 m (min.) below channel bottom. Design piles as columns if footing bottom is above total scour depth for Q_{max}.*
        - *Friction Piles*
          - *Attain design capacity below Q_{100} total scour depth.*
        - *End Bearing Piles*
          - *Minimum pile tip elevation 6.0 m below Q_{100} total scour depth.*
    - *On Soil*
      - *Place bottom of footing 3.0 m (min.) below channel bottom and below total scour depth for Q_{100}.*

Notes:

1. *Check adequacy under dead load only at the greater of Q_{100} or $Q_{overtopping}$.*
2. *Total scour = contraction scour + local scour.*
3. *The project Geotechnical Report will determine the final footing configuration.*

FOUNDATION DESIGN CRITERIA
(Intermediate Bents)

Figure 20.1B

20.2 SPREAD FOOTINGS AND PILE CAPS

Spread footings are typically thick, reinforced concrete slabs sized to meet the structural and geotechnical loading requirements for the proposed structural system. Spread footings are used to support piers, bents, abutments and retaining walls where suitable soils or rock are located at a relatively shallow depth. Suitable material is usually construed as being material where the last two blow counts are 35 or greater on a standard SPT test at a depth of less than 3 m. Factors affecting the size of the footings are the structural loading versus the ability of the soil to resist the applied loads.

Where suitable materials lie below the depth that can be excavated economically or where no firm layers were identified in the subsurface exploration, a deep foundation may be used. Piles and drilled shafts are the most common types of deep foundations used in Montana. See Section 20.3 and 20.4 for further discussion on piles and drilled shafts.

Deep foundations typically use an intermediate member called a pile cap and piles or drilled shafts to transfer the structural loads to strata that is capable of resisting the loads. Pile caps appear similar to spread footings but differ in that they transmit the loads to piles or drilled shafts instead of directly to the soil below the cap.

20.2.1 Minimum Dimensions/Materials

The following criteria shall apply:

1. Footing and Cap Thickness:
   a. Spread Footings: 600 mm
   b. Pile Caps: 750 mm
2. Class of Concrete: Typically Class DD, except for underwater placements where Class DS is used.
3. Compressive Strength: 28 day (for structural design):
   a. DD: $f'_c = 21$ MPa
   b. DS: $f'_c = 17$ MPa
4. Reinforcing Steel: $f_y = 420$ MPa

20.2.2 Footing Thickness and Shear Design

Reference: LRFD Articles 5.8.3, 5.13.3.6 and 5.13.3.8

The footing thickness may be governed by the development length of the footing dowels (footing to wall or column) or by concrete shear requirements. Generally, shear reinforcement in footings should be avoided. If concrete shear governs the thickness, it is usually more economical to use a thicker footing unreinforced for shear instead of a thinner footing with shear reinforcement. Footing thicknesses will be increased in 50-mm increments.

20.2.3 Depth and Cover

Reference: LRFD Articles 2.6.4.4.2 and 10.6.1.2

The vertical location of a footing should satisfy the following criteria. These criteria are summarized in Figure 20.1B.

20.2.3.1 Bottom of Footings

The following applies:

1. Footings on Soil. The bottom of footings on soil shall be set at least 3.0 m below the channel bottom and below the total scour depth determined for Q_{100}.
2. Footings on Rock. Small embedments (keying) should be avoided because blasting to achieve keying frequently damages and renders it the sub-footing rock structure more susceptible to scour. If footings on

smooth massive rock surfaces require lateral restraint, steel dowels should be drilled and grouted into the rock below the footing level. The bottom of the footings should be at least 0.9 m below the surface of scour-resistant rock with the top of the footings at least below the rock surface.

3. Footings on Erodible Rock. Weathered or other potentially erodible rock formations need to be carefully assessed for scour. The Geotechnical Section should be consulted for the spread footing foundation. The decision should be based upon an analysis of intact rock cores, including rock quality designations and local geology, hydraulic data and anticipated structure life. An important consideration may be the existence of a high-quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth for the design flood for scour should be estimated and the footing base located so that the top of the footing is below the estimated contraction plus local scour. The excavation above the top of the spread footing is usually backfilled with the same material that was excavated.

4. Footings Placed on Tremie Seals and Supported on Soil. The location of the top of the footing to be placed on a seal is determined in the same manner as a footing placed directly on the ground. That is, the bottom of the footing is below the estimated scour depth at the design flood. The elevation at the bottom of the footing is the same as the top of the seal. The required seal depth is then calculated assuming that the contractor will have to dewater the cofferdam to place the footing "in the dry." The seal mass counteracts the buoyant forces that occur when the cofferdam is dewatered. This depth is typically 40% of the head from the bottom of the seal to the normal water elevation. This 40% is simply the ratio of $\gamma_{water}/\gamma_{concrete}$. To help accommodate construction uncertainties while locating the cofferdam in the channel, the length and width of the seal are 1 m greater than the dimensions of the footing. This allows for minor "adjustments," if necessary, to position the footing for the pier correctly.

20.2.3.2 Top of Footings

The top of the footing on dry land shall have a minimum of a 300-mm permanent earth cover.

20.2.4 Bearing Resistance and Eccentricity

Reference: LRFD Article 10.6.3

For spread footings, the Geotechnical Section will provide the factored nominal bearing resistance to the Bridge Bureau. (The nominal bearing resistance is what was traditionally called the allowable bearing capacity.) The maximum factored design bearing pressure is shown on the Structural Plans for the footing.

20.2.4.1 Soils under Footings

Reference: LRFD Article 10.6.3.1.5

In contrast to the approach in the former Standard Specifications, a reduced effective footing area based upon the calculated eccentricity is used to include these effects. Uniform design bearing pressure is assumed over the effective area. This uniform-pressure model acknowledges the plastic nature of soil. An example is provided in Figure 20.2C.

The location of the resultant of the center of pressure based upon factored loads should be within the middle ½ of the base.

20.2.4.2 Rock

Reference: LRFD Article 10.6.3.2.5

Following the traditional approach, a triangular or trapezoidal pressure distribution is assumed

for footings on rock. This model acknowledges the linear-elastic response of rock.

The location of the resultant center of pressure based upon factored loads should be within the middle ¾ of the base.

20.2.5 Sliding Resistance

The approximate coefficients of friction in Figure 20.2A may be used to check the sliding resistance unless more exact coefficients are established for a particular case.

Keys in footings to develop passive pressure against sliding are not very effective and their economic justification is often over estimated. However, when it becomes necessary to use a key, the designer shall submit studies to the Bridge Area Engineer during preliminary design.

Concrete On:	Coefficient of Friction
Wet Clay or Silty Clay	0.33
Sand, Silty or Clay Gravel	0.40
Coarse Grain Soil with Silt	0.45
Dry Clay	0.50
Coarse Grain Soil without Silt	0.55
Gravel and Sand	0.60
Sound Rock or Concrete	Place footings against rock

Reference: 1978 MDT Structures Design Manual.

SLIDING RESISTANCE
Figure 20.2A

20.2.6 Settlement

Reference: LRFD Articles 3.12.6, 10.6.2.2 and 10.7.2.3

Differential settlement (SE) is considered a load in the LRFD Specifications. Generally, due to the methods used to determine allowable foundation loads by MDT, force effects due to differential settlement need not be investigated. If varying conditions exist, settlement will be addressed in the Geotechnical Report and the following effects should be considered:

1. Structural. The differential settlement of substructures causes the development of force effects in continuous superstructures. These force effects are directly proportional to structural depth and inversely proportional to span length, indicating a preference for shallow, large-span structures. They are normally smaller than expected and tend to be reduced in the inelastic phase. Nevertheless, they may be considered in design if deemed significant, especially those negative movements which may either cause or enlarge existing cracking in concrete deck slabs.

2. Joint Movements. A change in bridge geometry due to settlement causes movement in deck joints which should be considered in their detailing, especially for deep superstructures.

3. Profile Distortion. Excessive differential settlement may cause a distortion of the roadway profile that may be undesirable for vehicles traveling at high speed.

4. Appearance. Viewing excessive settlement may create a feeling of lack of safety.

20.2.7 Reinforcement

Reference: LRFD Articles 5.10.8 and 5.13.3

Unless other design considerations govern, the reinforcement in footings should be as follows:

1. Longitudinal Steel. Place longitudinal distribution bars on top of the primary transverse steel for the top mat of footing reinforcement. Place the transverse steel on top of the longitudinal steel for the bottom mat of footing reinforcement.

2. Embedment Length. Bar embedment lengths shall be shown on the plans. In spread footings, hooks may be omitted on transverse footing bars unless bond calculations dictate otherwise.

   Vertical steel extending upwards out of the footing shall also extend down to the bottom footing steel and shall be hooked on the bottom end regardless of the footing thickness.

3. Spacing. In spread footings, the spacing of reinforcing steel shall not exceed 150 mm in either direction.

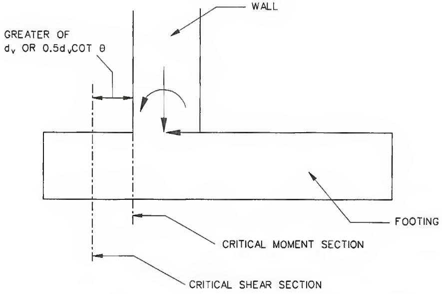
4. Other Reinforcement Considerations. Article 5.13.3 in the LRFD Specifications specifically addresses concrete footings. For items not included, the other relevant provisions of Section 5 should govern. For narrow footings, to which the load is transmitted by walls or wall-like piers, the critical moment section shall be taken at the face of the wall or pier stem and the critical shear section a distance equal to the larger of "d_v" (effective shear depth of the footing) or "0.5 $d_v\cot\theta$" from the face of the wall or pier stem where the load introduces compression in the top of the footing section. See Figure 20.2B. For other cases, either Article 5.13.3 is followed, or a two-dimensional analysis may be used for greater economy of the footing.

20.2.8 Joints

Footings do not generally require expansion joints. Where used, footing construction joints should be offset 600 mm from expansion joints or construction joints in walls and should be constructed with 75-mm deep keyways placed in the joint.

20.2.9 Stepped Footings

The difference in elevation of adjacent stepped footings should not be less than 150 mm. The lower footing should extend 600 mm under the adjacent higher footing, or an approved anchorage system may be used.



CRITICAL SECTIONS FOR MOMENT AND SHEAR FOR WALLS OR WALL-LIKE PIERS

Figure 20.2B

20.2.10 Additions to Existing Footings

At the interface between existing and new footings, existing concrete should be removed as needed to provide adequate development length for lap splicing of existing reinforcement, or an approved anchorage system may be used.

Where the substructure of an existing structure is extended, the old footing with respect to the new footing should be shown on the new Footing Plan Sheet.

20.2.11 Cofferdams

The purpose of a cofferdam is to provide a protected area within which an abutment or a pier can be built. A cofferdam in general is a structure consisting of steel sheeting driven into the ground and below the bottom of the footing elevation and braced to resist pressure. It should be nearly watertight and capable of being dewatered.

Generally, cofferdams are designed and detailed by the Contractor and reviewed by the Construction Bureau.

20.2.12 Field Integrity Testing

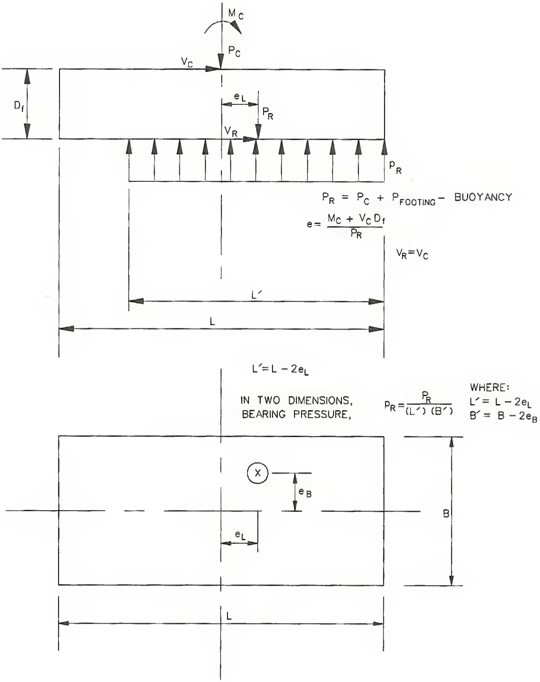
All excavations for spread footings are tested to check the integrity of the subsoil and to determine if it is necessary to adjust the footing elevations.

20.2.13 Design Example of Analysis of a Spread Footing on Competent Soil

See Figure 20.2C for a schematic example of a footing on soil to support an interior pier at a stream crossing.

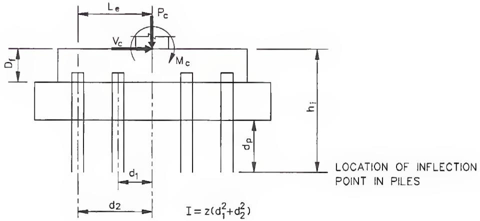
20.2.14 Design Example of Analysis of Pile-Supported Footings

See Figure 20.2D for a schematic example of the analysis of a pile-supported footing to support an interior pier at a stream crossing (fixed pile connection). See Figure 20.2E for a similar footing assuming a pinned pile connection.



ANALYSIS OF SPREAD FOOTING ON COMPETENT SOIL

Figure 20.2C



Assumptions: Pile cap is rigid. Pile connections are fixed and shear forces per pile are significant.

Footing is considered rigid if $L_e/D_f \leq 2.2$
$P_R = P_c + P_{footing} + P_{seal}$ – Buoyancy

To obtain forces in piles, sum moments about inflection point:

$$P_{max} = \frac{P_R}{\#\text{ of piles}} + \frac{(V_c h_i + M_c)\, d_2}{I_z}$$

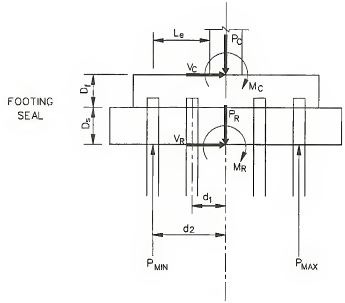
$$P_{min} = \frac{P_R}{\#\text{ of piles}} - \frac{(V_c h_i + M_c)\, d_2}{I_z}$$

$$V_{pile} = \frac{V_c}{\#\text{ piles}}$$

$$M_{pile} = V_{pile}\, d_p$$

PILE FOOTING ANALYSIS
(Fixed Pile Connection)

Figure 20.2D



Assumptions: Pile cap is rigid. Pile connections are pinned or shear force in pile is small.

$P_R = P_c + P_{footing} + P_{seal} - \text{Buoyancy}$

$V_R = V_c - V_{\text{passive soil pressure on footing and seal}}$ *Note: Passive soil pressure is typically ignored.*

$M_R = M_c + V_c\,(D_f + D_s)$

Footing is considered rigid if $L_e/D_f \leq 2.2$

Pile Loads:

$$P_{max} = \frac{P_R}{\#\,\text{piles}} + \frac{M_R\ d_2}{\Sigma d_i^2}$$

$$P_{min} = \frac{P_R}{\#\,\text{piles}} - \frac{M_R\ d_2}{\Sigma d_i^2}$$

PILE FOOTING ANALYSIS
(Pinned Pile Connection)

Figure 20.2E

20.3 PILES

20.3.1 Pile Selection and Design

Where underlying soils provide inadequate bearing capacity or excessive settlement, piles may serve to transfer loads to deeper suitable strata. Piles may function through skin friction and/or through end bearing. Required bearing capacity, soil conditions and economic considerations determine pile type. MDT traditionally uses steel pipe piles, steel H-piles and fluted steel piles.

The following applies to piles:

1. Verify the design of all piles by engineering analysis.
2. Unless project conditions require otherwise, use one pile size and type throughout a project. Mixing pile types or sizes on a project increases cost and the likelihood of construction errors.
3. Require a two-component epoxy paint meeting the requirements of the **Standard Specifications** to protect any part of the pile exposed to the environment. The paint must extend at least 600 mm below the channel bottom or ground line.
4. Preliminary Design Assumption. For preliminary design purposes only, select the number of piles on the basis of allowing a maximum service-load stress of 62.0 MPa.

20.3.2 Types

20.3.2.1 Steel Pipe Piles

Reference: LRFD Articles 6.9.5 and 6.12.2.3

In addition to the information contained in the LRFD references, the following applies to the use of steel pipe piles in MDT projects:

1. Usage. Steel pipe piles may serve as bearing piles, as friction piles, or as a combination of the two.
2. Diameter. MDT normally uses pipe piles either 406 mm or 508 mm in diameter, with a wall thickness of 12.7 mm in either case. MDT may accept other sizes with prior approval from the Bridge Area Engineer.
3. Concrete Fill. The contractor will fill each pipe pile with Class DD concrete after driving with concrete having a compressive strength of at least $f_c' = 21.0$ MPa.
4. Neglect Concrete for Design. Except in rare, extreme design problems, MDT does not include the concrete in analyzing steel pipe pile capacity for design. Design the pipe pile without the concrete. The concrete will provide additional conservation to the design.

20.3.2.2 Steel H-Piles

The following will apply to steel H-piles:

1. Usage. These are generally used either where the pile obtains most of its bearing resistance from end bearing on rock or as recommended in the Geotechnical Report.
2. Size. Pile size designations may be HP310 or HP360; HP310 is typical.

20.3.2.3 Fluted Steel Piles

The following will apply to fluted steel piles:

1. Usage. Fluted steel piles are generally used only in deep, soft materials.
2. Size. The gage of the pile wall thickness may be 9 (3.8 mm), 7(4.6 mm), 5 (5.3 mm) or 3 (6.1 mm) gage.

20.3.3 Pile Length

Reference: LRFD Articles 10.7.1.10, 10.7.1.11 and 10.7.1.12

If a pile foundation is determined to be the appropriate solution to the structural and geotechnical specifics at the site, the length of the piles will be estimated based on information in the Geotechnical Report. The following is provided to guide the designer through the decision-making process in determining pile length:

1. Minimum Length. In special cases, it will be necessary to specify the minimum length of piles in the plans. Piles should be a minimum of 3.0 m in length and, unless refusal is encountered, penetrate into hard cohesive or dense granular original soil not less than 3.0 m. If the depth to suitable rock strata is less than 3.0 m, MDT practice is to seat the pile in holes cored in the rock. A minimum core depth of 1.0 m into scour-resistant rock is recommended. Where piles less than 3.0 m in length are anticipated, consideration shall also be given to lowering the elevation of the bottom of footing and providing spread footings instead.
2. Tip Elevation for Friction Piles. Show the minimum pile tip elevation from the Geotechnical Report on the drawing of the structural element.
3. Tip Elevation for Point Bearing Piles. Show the minimum pile tip elevation from the Geotechnical Report on the drawing of the structural element. The bottom of the tip is usually placed some distance into the formation material to ensure that it is through any weathered surficial material and into competent rock.
4. Pile Tip Elevation Guidelines. Figure 20.1B provides guidance for use in determining minimum pile tip elevations.

20.3.4 Design Details

Reference: LRFD Article 10.7.1

The following will apply to the design of piles:

1. Battered Piles. The use of battered piles must be justified by analysis. When used, a pile batter of 12 vertical to 2 horizontal is considered desirable. However, piles may be battered to a maximum of 4 vertical to 1 horizontal where substantial resistance is not otherwise attainable. For the outside row of piles in footings, a batter should be provided on alternating piles. Where closely spaced battered piles are used, the pile layout should be checked to ensure that battered piles do not intersect. Battered piles should not be employed where extensive downdrag load is expected because this load causes flexure in addition to axial force effects. Battered piling can not be used within cofferdams.
2. Spacing. Spacing of piles is specified in Article 10.7.1.5 in the LRFD Specifications. Center-to-center spacing should not be less than the greater of 750 mm or 2.5 times the pile diameter or width of pile. The distance from the side of any pile to the nearest edge of footing shall be greater than 250 mm.
3. Embedment. Embed piles a minimum of 500 mm into the footing after all damaged pile material has been removed. If pile reinforcement is extended into the footing, satisfying the provisions of LRFD Article 5.13.4.1, the embedment length may be reduced. Pile connections with high tensile loads or moments require additional design considerations.
4. Downdrag (DD) Loads. When a pile penetrates a soft layer subject to settlement, the force effects of downdrag or negative loading on the foundations must be evaluated. Downdrag acts as an additional permanent axial load on the pile. If the force is of sufficient magnitude, structural failure of the pile or a bearing failure at the

tip is possible. At smaller magnitudes of downdrag, the pile may cause additional settlement. For piles that derive their resistance mostly from end bearing, the structural resistance of the pile must be adequate to resist the factored loads including downdrag. Battered piles should be avoided where downdrag loading is possible due to the potential for bending of the pile. Downdrag forces can be mitigated by preboring and filling the prebored hole with pea gravel, or by building the approach fill far enough in advance of the pile driving for the fill to settle out.

5. Uplift Forces. Uplift forces can be caused by lateral loads, buoyancy or expansive soils. Piles intended to resist uplift forces should be checked for resistance to pullout and structural resistance to tensile loads. The connection of the pile to the footing must also be checked.

6. Laterally Loaded Piles. The resistance of laterally loaded piles must be estimated according to approved methods. Several methods exist for including the effects of piles and surrounding soil into the structural model for lateral loadings including seismic loads. These methods are discussed in Section 20.4.2.

7. Group Effect. Minimum spacing requirements are not related to group effect. Group effects are specified in the LRFD Specifications in Article 10.7.3.7.3 and in Article 10.7.3.10.

8. Pile Tips. Use pile tips to minimize damage to the piles.

9. Pile Loads. The pile load shall be shown in the Plans. This information will help ensure that pile driving efforts during the construction process will result in a foundation adequate to support the design loads. The load to which piles are to be driven shall be greater than or equal to the total factored load. The governing strength limit state load combination from LRFD Table 3.4.1-1 shall also be indicated. Pile design loads are typically limited to less than 900 kN to help maintain the competition among local contractors, who would otherwise be forced to rent larger equipment if they had to drive to higher pile capacities.

10. Pile Load Tests. Where pile design loads are high or where the pile quantity is large, pile load tests may be justified. The designer should consult MDT's Geotechnical Section if considering pile load testing. Test locations should be shown in the plans or described in the special provisions.

20.4 DRILLED SHAFTS

Reference: LRFD Article 10.8

20.4.1 Design

The following will apply to the design of drilled shafts:

1. Usage. Drilled shafts may be an economical alternative to driven piles. Drilled shafts should also be considered to resist large lateral or uplift loads when deformation tolerances are relatively small. Drilled shafts derive load resistance either as end-bearing shafts transferring load by tip resistance or as friction shafts transferring load by side resistance.

2. Socketed Shafts. A schematic drawing of a rock-socketed shaft is shown in Figure 20.4A. Where casing through overburden soils is required, design the shaft as one size and do not step down when going into formation material.

3. Column Design. Because even soft soils provide sufficient support to prevent lateral buckling of the shaft, it may be designed according to the criteria for short columns in Article 5.7.4.4 of the LRFD Specifications. If the drilled shaft is extended above ground to form a pier or part of a pier, it should be analyzed and designed as a column. The effects of scour around the shafts must be considered in the analysis.

4. Reinforcement. The shaft will have a minimum of 0.8 percent of the gross concrete area and will extend from the bottom of the shaft into the footing. If the drilled shaft is extended above ground level, reinforcement should satisfy the requirements of Article 5.7.4.2 in the LRFD Specifications.

20.4.2 Pile and Drilled Shaft Modeling

Several possibilities exist for including the effects of piles and surrounding soil into the structural model for lateral loadings including seismic loads. Two of these methods are summarized in Figure 20.4B and include:

1. equivalent cantilever model, and
2. equivalent soil springs model.

The simplest approach is to assume that an equivalent cantilever column can be used to model the pile. The section of the cantilever is the same as that of the pile but its length (depth to "fixity") is adjusted so as to give either the same stiffness at ground level or the same maximum bending moment as in the actual soil-pile system.

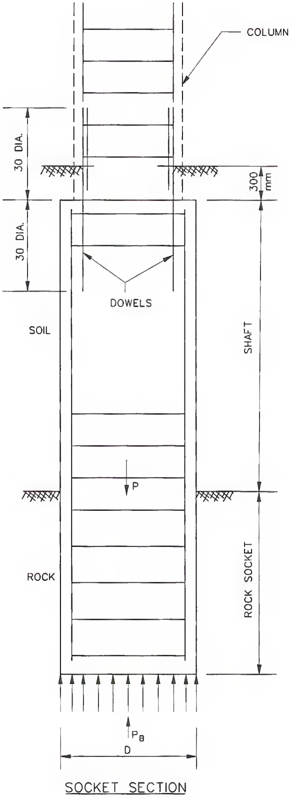
The length to fixity of the equivalent cantilever can be determined from charts such as those in Figures 20.4C and 20.4D, which are for large diameter concrete piles, or from equations relating the stiffnesses of the pile and soil given in Figure 20.4E. The soil constants, K_h and n_h, for use in the equations are subsequently given in Figure 20.4F.

In most cases, the use of either the charts or the relative stiffness formulation will give satisfactory results, eliminating the need for a detailed foundation model. Note that the charts give only the effective depth for stiffness considerations, and pile moments based on this length will be overestimated. It should also be noted that the two methods (charts, relative stiffnesses) give different results for the effective depth to fixity. This is in part a reflection of the uncertainty associated with foundation engineering. However, both methods provide a rational and simple way for including foundation flexibility in the analysis of bridges, and results using either method will be closer to the actual behavior than will results from a model which rigidly fixes the bridge at ground level.

Typical ranges for the effective length to fixity (for stiffness) are from 3 to 9 pile diameters, the

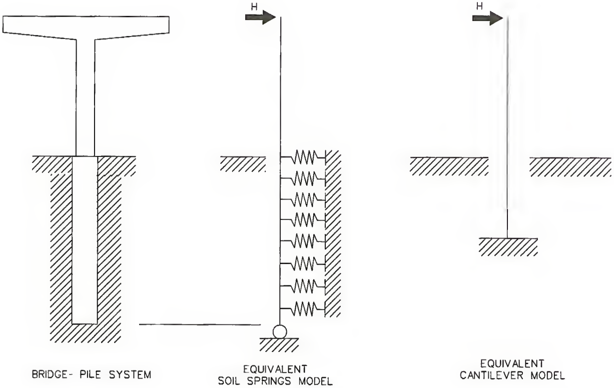
low end of the range being for very stiff sites. It should be noted that this depth to fixity is potentially a function of the direction of loading, because pile group effects may be different longitudinally and transversely. In the absence of more specific information, the effective modulus of horizontal subgrade reaction (K_h) for each pile may be assumed to vary linearly from 25% of the K_h value for a single pile, when the spacing in the direction of load is 3 pile diameters, to the K_h value for a single pile, when the spacing is 8 pile diameters.

The second technique noted above involves the use of p-y curves to represent the soil. This is the equivalent soil springs model. The advantage of this approach is the avoidance of the need to calculate equivalent spring constants as in the above method. The disadvantage is the substantial increase in the size and complexity of the structural model. The solution's accuracy is primarily a function of the spacing between nodes used to attach the soil springs to the pile (the closer the spacing, the better the accuracy), and is not so dependent on the pile itself. Simple beam column elements are usually adequate for modeling the pile behavior. The computer program LPILE is used by MDT to model equivalent soil springs.



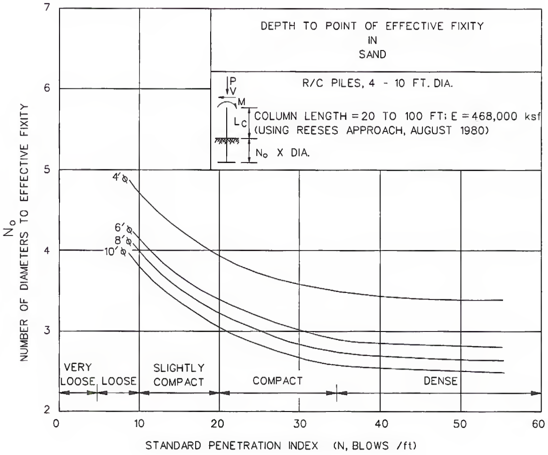
DRILLED SHAFTS

Figure 20.4A



METHODS OF REPRESENTING PILE FOUNDATION STIFFNESS

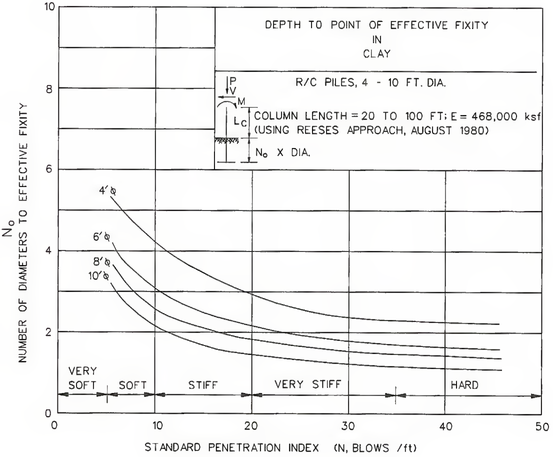
Figure 20.4B



0-28	28-30	30-36	36-41	APPROXIMATE ϕ IN DEGREES

DEPTH TO POINT OF EFFECTIVE FIXITY FOR DRILLED SHAFTS IN SAND

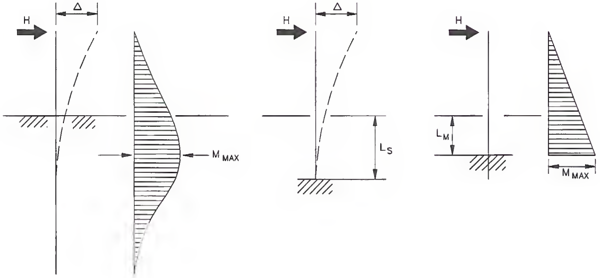
Figure 20.4C



0-0.5	0.5-1	1-2	2-4	4-6 SHEAR STRENGTH, ksf

DEPTH TO POINT OF EFFECTIVE FIXITY FOR DRILLED SHAFTS IN CLAY

Figure 20.4D



	L_S	L_M
Cohesive Soil Constant (K_h)	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless Soil Constant (n_h)	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

EQUIVALENT CANTILEVERED METHOD USING RELATIVE STIFFNESS FACTORS

Figure 20.4E

Soil Type	Site Data			Design Parameters			
	N (blows/ft)	Undrained Shear Strength (ksf)	ϕ' (degrees)	n_h (kips/ft^3)		K_h (kip/ft^2)	E_S (kip/ft^2)
Cohesionless Soils							
- Dense	30-50		45	100	60		
- Loose	4-10		30	15	6		
Cohesive Soils							
- Hard	20-60	3-15				375	520
- Medium	8-15	1-2				125	170
- Soft	2-4	0.3-0.6				30	40

N = standard penetration test resistance

K_h = modulus of horizontal subgrade reaction

n_h = constant of horizontal subgrade reaction $= \frac{dK_h}{d_z}$

E_s = soil modulus of elasticity

ϕ' = effective soil internal angle of friction

SUGGESTED SOIL STIFFNESS PARAMETERS FOR PRELIMINARY SEISMIC ANALYSIS

Figure 20.4F

Table of Contents

Chapter Twenty-one
HIGHWAY BRIDGES OVER RAILROADS

21.1 PROCEDURES

21.1.1 Utilities Section

The MDT Utilities Section within the Right-of-Way Bureau is responsible for coordinating with the railroad companies where MDT projects impact railroads. The Utilities Section's responsibilities include:

1. obtaining cost estimates for securing agreements with railroad companies for the relocation and adjustment of their facilities as required for highway construction; and

2. conducting direct negotiations with railroad companies, when necessary, to secure temporary access or to acquire portions of their operating rights-of-way for highway purposes.

21.1.2 Project Development

Because of the unique nature of highway-railroad grade separations, special coordination must occur when a railroad alignment and a road alignment intersect. A preliminary layout is developed giving consideration to the minimum horizontal and vertical clearances in this Chapter. The Bridge Area Engineer and Utilities Section will schedule an on-site meeting at the bridge site with the impacted railroad companies. The Bridge Area Engineer will submit the preliminary bridge layout to the Utilities Section for submission to the railroad company before this meeting. The on-site meeting should evaluate railroad considerations, which include:

1. construction of future tracks;

2. off-track maintenance roadways;

3. longitudinal drainage requirements, from both the railroad's perspective and the MDT's perspective;

4. heavy snow areas; and

5. bent locations.

See Chapter Two of the **MDT Structures Manual** for a discussion on how the coordination with railroad companies is incorporated into the project development process for a bridge project.

21.2 DESIGN CRITERIA

21.2.1 General

Highway bridges constructed over railroads must be designed to be consistent with the geometric requirements of railroads. This includes criteria for lateral clearances, vertical clearances and railroad structure width. These criteria are based on:

1. the Federal Highway Administration participation limits for railroad geometrics;
2. the requirements of the State of Montana;
3. the specifications of the American Railroad Engineering and Maintenance-of-Way Association (AREMA); and
4. the criteria established by individual railroad companies.

The following railroad companies operate in the State of Montana:

1. Union Pacific (UP);
2. Burlington Northern and Santa Fe (BNSF);
3. Montana Rail Link (MRL);
4. Central Montana Rail, Inc. (CMR);
5. Montana Western Rail; and
6. RARUS.

21.2.2 Basic Geometric Configuration

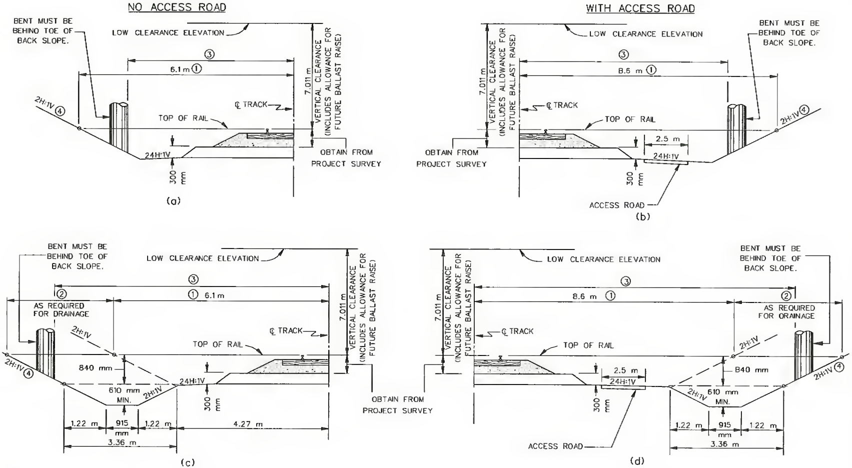
The basic geometric configuration of the railroad cross section passing under a highway bridge is primarily based on the following factors:

1. Type of Rail Line. This may be either a mainline or an industrial/branch line.
2. Number and Type of Tracks. Multiple sets of railroad tracks obviously require a longer bridge.
3. Drainage Treatments. The railroad typical section may or may not include a parallel drainage ditch.
4. Access Road. The railroad typical section may or may not include parallel off-track equipment/maintenance roads for access.
5. Specific Requirements of Individual Railroad Companies. The different companies have varying requirements for subgrade width and ballast thickness.
6. Backslope. This will be 2H:1V maximum unless it is specifically engineered for stability at a steeper slope and slope protection is provided.

Figures 21.2A and 21.2B present the basic railroad cross sections based on these variables. Sections 21.2.3 and 21.2.4 present additional information which must be considered. Meeting the clearance requirements in Figures 21.2A and 21.2B typically determines the length and cost of the highway structure.

21.2.3 Montana/AREMA Recommendations

The Montana Public Service Commission (PSC) promulgates regulations for railroad companies operating in the State of Montana. In addition, the American Railroad Engineering and Maintenance-of-Way Association (AREMA) publishes the **AREMA Manual**, which provides design standards for railroads. This Section presents those PSC and AREMA regulations, specifications and recommendations that apply to the railroad cross section under a highway bridge.



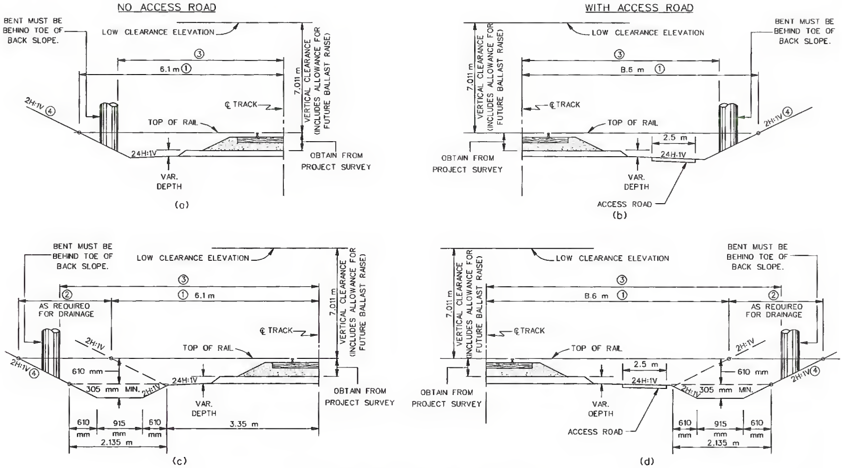
① To exceed 6.1 m (without access road) or 8.6 m (with access road), the need for heavy snow areas and/or longitudinal drainage along the track must be documented.

② An on-site meeting with railroad owners/operators will be scheduled soon after receipt of survey to discuss drainage needs.

③ Face of bents will be a minimum of 7.7 m from the centerline of a mainline track. Bents less than 7.7 m from the centerline of a railroad track shall be of heavy construction or shall be protected by a reinforced concrete crashwall. See Section 21.2.3.4. At multiple track installations, at railroad yards or where an overpass is being rehabilitated or widened, the horizontal clearances will be determined at the initial meeting with the railroad involved.

④ Slope to fit soil type but not steeper than a 2H:1V backslope, unless a reinforced embankment is designed. See Section 21.2.3.5.

RAILROAD CLEARANCES (Mainline)
Figure [illegible]2A



① To exceed 6.1 m (without access road) or 8.6 m (with access road), the need for heavy snow areas and/or longitudinal drainage along the track must be documented.

② An on-site meeting with railroad owners/operators will be scheduled soon after receipt of survey to discuss drainage needs.

③ Face of bents will be a minimum of 7.7 m from the centerline of a mainline track. Bents less than 7.7 m from the centerline of a railroad track shall be of heavy construction or shall be protected by a reinforced concrete crashwall. See Section 21.2.3.4. At multiple track installations, at railroad yards or where an overpass is being rehabilitated or widened, the horizontal clearances will be determined at the initial meeting with the railroad involved.

④ Slope to fit soil type but not steeper than a 2H:1V backslope, unless a reinforced embankment is designed. See Section 21.2.3.5.

RAILROAD CLEARANCES (Industrial/Branch Line)
Figure 21.2B

21.2.3.1 Regulatory Template for Tangent Sections of Track

Figure 21.2C represents a compilation of information from several sources and is the template of the general outline for tangent track as defined by PSC regulations. This must be considered an inviolable railroad template for permanent construction, although at least one RR (UPRR) indicates that it is willing to negotiate a reduced vertical clearance of 6.4 m during construction.

21.2.3.2 Horizontal Curves

The clearances shown in Figures 21.2A and 21.2B are for tangent track. On curved track, the lateral clearances on each side of the track centerline shall be increased 38.1 mm per degree of curvature on the railroad alignment. When the fixed obstruction is on tangent track but the track is curved within 24.384 m of the obstruction, the lateral clearances on each side of track centerline shall be increased as shown in Figure 21.2D.

On superelevated track, the track centerline remains perpendicular to a plane across the top of rails. The superelevation of the outer rail shall be in accordance with the recommended practice of AREMA.

Distance from Obstruction to Curved Track (m)	Increase Per Degree of Curvature (mm)
6.096	38.100
12.192	28.575
18.288	19.050
24.384	9.525

Note: To convert degree of curvature (D) to radius of curve (R, in meters), R = 1746.8/D.

LATERAL CLEARANCE INCREASE FOR FIXED OBSTRUCTION

Figure 21.2D

21.2.3.3 Track Centers

The following applies to the width of track centers:

1. Mainline Track. According to the PSC, the minimum distance between track centers is 4.27 m with minor exceptions. Individual railroad companies may have different requirements. These requirements, and any plans the railroad companies may have for future additional tracks, must be discussed at the on-site meeting with the railroad.

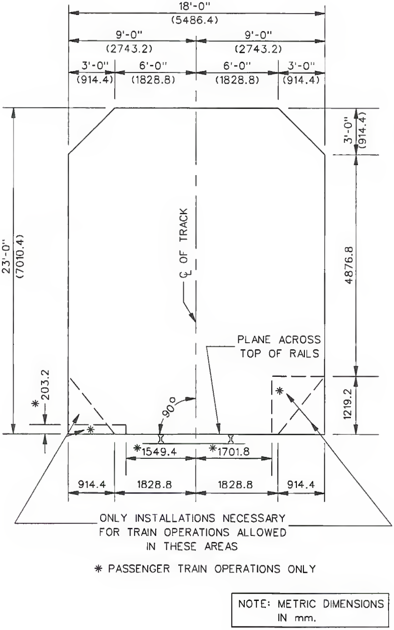
2. Parallel Team, House or Industry Track. The minimum distance between track and centers shall be 3.96 m.

3. Ladder Tracks. Ladder tracks shall be at least 6.1-m centers from any other parallel track.

21.2.3.4 Pier Protection

To limit damage by the redirection and deflection of railroad equipment, piers supporting bridges over railways and with a clear distance of less than 7.7 m from the centerline of a railroad track shall be of heavy construction (defined below) or shall be protected by a reinforced concrete crash wall. Crash walls for piers from 3.66 m to 7.7 m clear from the centerline of track shall have a minimum height of 1.85 m above the top of rail. Piers less than 3.66 m clear from the centerline of track shall have a minimum crash wall height of 3.66 m above the top of rail. The following also applies:

1. The crash wall shall be at least 770 mm thick and at least 3.66 m long. When two or more columns compose a pier, the crash wall shall connect the columns and extend at least 305 mm beyond the outermost columns parallel to the track. The crash wall shall be anchored to the footings and column, if applicable, with adequate reinforcing steel and shall extend to at least 1.22 m below the lowest surrounding grade.



REGULATORY TEMPLATE FOR TANGENT TRACK

Figure 21.2C

2. Consideration may be given to providing protection for bridge piers over 7.7 m from the centerline track as conditions warrant. In making this determination, consider such factors as horizontal and vertical alignment of the track, embankment height and an assessment of the consequences of serious damage in case of a collision.

21.2.3.5 Side Slopes

When slopes steeper than 2H:1V are proposed, some method of preventing the slopes from eroding must be provided. Methods could include slope protection with concrete or asphalt or a very wide drainage ditch. Seeding and attempting to develop cover vegetation are not effective techniques for preventing erosion of slopes steeper than 2H:1V.

21.2.3.6 Existing Tracks

Clearances for reconstruction work or for alteration of existing tracks are dependent on existing physical conditions and, where reasonably possible, should be improved to meet the requirements for new construction.

21.2.4 Railroad Company Design Criteria

Each railroad company operating within the State of Montana requests or requires that MDT comply with its design criteria for highway bridges over railroads. These may apply to restrictions during construction, fencing, drainage, erosion control, etc. The bridge designer must ensure that the Department coordinates early with the railroad companies, through the MDT Utilities Section, to identify these site-specific design criteria.

21.2.5 FHWA Participation

The Appendix to Subpart B of 23 CFR 646 presents the limits of FHWA participation for the costs of highway bridges over railroads. For convenience, this Section reproduces those portions of the Appendix which are applicable in Montana.

The following implements provisions of 23 CFR 646.212(a)(3).

21.2.5.1 Lateral Geometrics

A cross section with a horizontal distance of 6.1 meters, measured at right angles from the centerline of track at the top of rails, to the face of the embankment slope, may be approved. The 6.1-m distance may be increased at individual structure locations as appropriate to provide for drainage if justified by a hydraulic analysis or to allow adequate room to accommodate special conditions, such as where heavy and drifting snow is a problem. The railroad must demonstrate that this is its normal practice to address these special conditions in the manner proposed. Additionally, this distance may also be increased up to 2.5 meters as may be necessary for off-track maintenance equipment, provided adequate horizontal clearance is not available in adjacent spans and where justified by the presence of an existing maintenance road or by evidence of future need for such equipment. All piers should be placed at least 2.8 meters horizontally from the centerline of the track and preferably beyond the drainage ditch. For multiple track facilities, all dimensions apply to the centerline of the outside track.

Any increase above the 6.1-m horizontal clearance distance must be required by specific site conditions and be justified by the railroad to the satisfaction of the MDT (SHA) and the FHWA.

21.2.5.2 Vertical Clearance

A vertical clearance of 7.1 m above the top of rails, which includes an allowance for future ballasting of the railroad tracks, may be approved. Vertical clearance greater than 7.1 m may be approved when the State regulatory

agency having jurisdiction over such matters requires a vertical clearance in excess of 7.1 m or on a site-by-site basis where justified by the railroad to the satisfaction of the MDT and the FHWA. A railroad's justification for increased vertical clearance should be based on an analysis of engineering, operational and/or economic conditions at a specific structure location.

21.2.6 Pedestrian Fencing

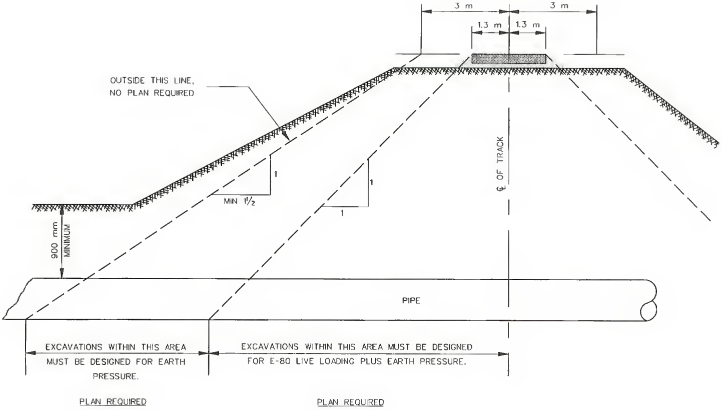
On bridges with sidewalks, designers must provide a means of protecting railroad facilities and the safety of railroad employees below from objects being thrown from above by pedestrians or passing motorists. Provide a 2.7-m high curved guard fence at least 8 m beyond the centerline of the outermost track or access road.

21.2.7 Shoring for Construction Excavations

Railroad companies will not allow any excavation that will cause settlement or warparge of their tracks. Each railroad company has a template adjacent to and below their tracks that limits excavations that may be made without shoring plans. Shoring plans must be an engineering submittal to the railroad for approval. The designer should make reasonable efforts to lay out the structure so that shored excavations are not needed for foundation construction. Figures 21.2E and 21.2F provide examples of the shoring templates for MRL and UPRR.

21.2.8 Control of Drainage from Highway Bridge Deck

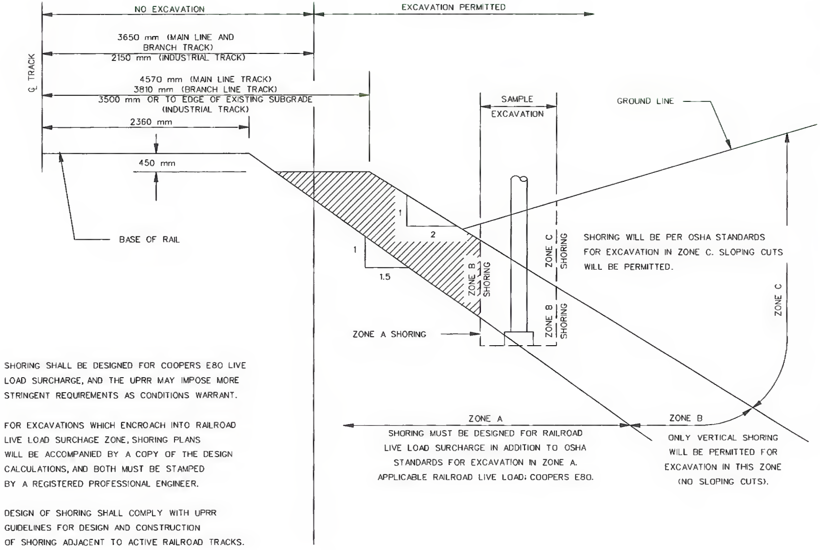
Railroad companies are quite sensitive to water that may come from the highway bridge onto their tracks or ballast. Drains are not permitted that would discharge water onto the track or access road areas at any time. Concrete splash blocks or aggregate ditch linings will be required at the discharge area of downspouts. Locate downspouts behind the face of the piers and make certain that they outflow to the drainage ditches.



MINIMUM COVER: ½ THE DIAMETER BELOW THE TOP OF THE TIE WITH A MINIMUM OF 610 mm BELOW THE BOTTOM OF THE TIE.

SHORING TEMPLATE
(MRL)

Figure 21.2E



SHORING TEMPLATE
(UPRR)

Figure 21.2F

Table of Contents

Table of Contents
(Continued)

Table of Contents
(Continued)

Section **Page**

Chapter Twenty-Two
BRIDGE REHABILITATION

The State of Montana contains more than 4500 bridges on its public roads and streets. Approximately 2600 of these are on the State highway system. Occasionally, these bridges require repair or rehabilitation which exceeds the scope of normal maintenance. In these cases, the bridge work is programmed as a capital improvement project. For the purpose of this Chapter, rehabilitation refers to:

1. restoring to a former state and/or capacity,

2. improving serviceability (structural and/or functional),

3. strengthening, and/or

4. widening.

22.1 SCOPE OF WORK

The Scope of Work for a bridge rehabilitation project typically meets one of the descriptions in the following Sections. The **Montana Bridge Design Standards** should be referenced to assist in determining an appropriate Scope of Work. Traffic control is often the most expensive component of a bridge rehabilitation project, and the cost and difficulty of maintaining traffic must be considered when selecting an appropriate project Scope of Work.

If the estimated cost of a bridge rehabilitation project approaches or exceeds 50% of the cost of a new structure, an in-depth cost analysis must be completed. It should include life-cycle cost analysis, alternative methods and different levels of rehabilitation.

22.1.1 Safety

Safety work is performed with a roadway overlay or overlay and widening project, but it can be performed as a "stand-alone" bridge project to correct a specific safety problem. Safety work may include:

1. Bridge Rail. All bridge railing must comply with the MDT Bridge Rail Policy. See Section 22.6.

2. Anti-Skid Treatment for Decks. If the Skid Number of an existing bridge within the limits of a roadway project indicates a potential safety hazard, then this alone could warrant a bridge deck overlay, especially if there is a history of wet-weather accidents. See Section 22.3.3.

3. Widening. It may or may not be warranted to widen a bridge as part of a safety project within the limits of a roadway project. This will be based upon the roadway classification, traffic volumes, and the width of the existing bridge.

22.1.2 Minor Rehabilitation

Minor rehabilitation work will generally be the types of activities listed below and is often done with a roadway overlay or overlay and widening project. It is not, however, limited to these activities:

1. guard angles;
2. expansion joints;
3. deck seal (silane, HMWM, etc.);
4. spot painting of structural steel;
5. drains and drainage systems;
6. elevation adjustments;
7. repainting or overcoat painting; and/or
8. highway lighting upgrades.

It may or may not be warranted to widen a bridge as part of a minor rehabilitation project within the limits of a roadway project. This will

be based upon the roadway classification, traffic volumes and the width of the existing bridge.

22.1.3 Major Rehabilitation

Usually, major rehabilitation work requires more plan development time than the corresponding roadway plans for an overlay or overlay and widening project, and it may be necessary to develop the project as a "stand-alone" bridge project. Major rehabilitation may include one or more of the following activities.

22.1.3.1 Bridge Deck

Bridge deck work, within the context of a major rehabilitation project, may include:

1. Deck Replacement. If the condition of an existing deck warrants replacement, the Bridge Area Engineer will perform a benefit-cost analysis to determine if widening the bridge is justified to meet the Department's bridge width criteria for new bridges. This will be based on the roadway classification, traffic volumes and the existing bridge width.

2. Deck Overlays. When deck conditions warrant, and sufficient lead time exists to obtain the deck survey and prepare plans, a deck overlay can be performed within the limits of a roadway overlay or overlay and widening project. If sufficient lead time does not exist to match the road project schedule, the deck overlay will be pursued as a "stand-alone" project. It may or may not be warranted to widen a bridge as part of a bridge deck overlay project within the limits of a roadway project. This will be based upon the roadway classification, traffic volumes, and the existing width of the bridge.

22.1.3.2 Structure Condition Ratings

All structural elements shall be returned to a condition rating of at least (7) as defined by the Montana Bridge Inspection Program. See Section 22.2 for a description of the Program.

22.1.3.3 Scour Countermeasures

If scour countermeasures are the only required work, no consideration will be given to widening the bridge.

22.1.3.4 Miscellaneous

Any safety or minor rehabilitation work listed in Sections 22.1.1 and 22.1.2 may also be performed as a part of a major rehabilitation project.

22.1.4 Seismic Retrofit

All bridges will be screened for seismic requirements in accordance with the Bridge Bureau's Seismic Screening Procedure. See Section 22.4.5 for MDT policies and practices for seismic retrofitting of existing bridges.

22.1.5 Trusses

Criteria for the rehabilitation of existing bridge trusses are in the **Montana Bridge Design Standards**. Secure the Bridge Engineer's approval before proceeding with design if the width, vertical clearance or load capacity in the standards cannot be obtained.

Structures with historical significance require special consideration when determining if they can be rehabilitated. See Section 13.8 for a list of the bridges in Montana that are included in or eligible for the National Register of Historic Places.

22.2 BRIDGE INSPECTION/BRIDGE MANAGEMENT

Many of the bridge rehabilitation projects programmed by MDT are identified through the Department's bridge inspection and bridge management activities. Section 22.2 provides a brief discussion on these.

22.2.1 National Bridge Inspection Standards (NBIS)

The National Bridge Inspection Standards (NBIS), a nationwide inspection and inventory program, is intended to detect structural problems. The Federal Highway Administration has regulations that each State transportation department must meet.

The following presents a brief discussion on the operational requirements of the NBIS:

1. Frequency of Inspections. Each bridge must be inspected at regular intervals.

2. Qualifications of Personnel. The Federal regulation lists the minimum qualifications for all bridge inspection personnel.

3. Inspection Procedures and Reports. Each State must have systematic methods for conducting field inspections and reporting its findings.

4. Records. Each State must have a systematic means of entering, storing and retrieving all bridge inspection data. The records must meet the Federal requirements.

5. Ratings. All bridges are rated according to their load-carrying capacity. This includes both the Operating and Inventory Ratings (see Section 22.2.2 for definitions). This information assists in the posting, the issuing of special overload permits, and the scheduling for rehabilitation or replacement.

22.2.2 Definitions

The following definitions apply to the NBIS and its implementation:

1. Inventory Rating. The load level that can be safely resisted by a structure for an indefinite period of time.

2. Operating Rating. The maximum permissible load level to which the structure may be subjected.

3. Sufficiency Rating. A numerical value from 0% to 100% that indicates a bridge's overall sufficiency to remain in service. The Rating is calculated from the Structure Inventory and Appraisal (SI&A) data.

4. Health Index The health index model is a single integral indicator of the structural health of the bridge. This indicator is expressed as a percentage value, which may vary from 0%, which corresponds to the worst possible condition, to 100% in the best condition.

22.2.3 MDT Bridge Inspection Program

22.2.3.1 Responsibility

The Bridge Management Section is responsible for collecting, maintaining and reporting bridge inspection information and for ensuring that the MDT Bridge Inspection Program complies with the requirements of the NBIS.

22.2.3.2 Description

The Bridge Management Section has published the following to describe and implement the MDT Bridge Inspection Program:

1. **MDT Bridge Inspection Manual,**

2. **MDT Fracture-Critical Inspection Manual,** and

3. **"Guidelines for Underwater Inspection."**

22.2.3.3 Classification of Substandard Bridges

To be considered for either *structurally deficient* or *functionally obsolete* classifications, the first digit of the Inventory Route Type (5A) must be coded "1," and the NBI Bridge Length Indicator must be coded "Y" to indicate a major structure (> 6.1 m face to face of supporting abutments).

To receive funding through the Highway Bridge Replacement and Rehabilitation Program (HBRRP) structures must be Structurally Deficient or Functionally Obsolete and have a Sufficiency Rating (SR) of 80% or below. Structures with an SR of 0 to 49.9 are eligible for replacement, and structures 50 to 80 are eligible for rehabilitation unless otherwise approved by FHWA.

The Sufficiency Rating formula is a method of evaluating highway bridge data by calculating four separate factors (structural adequacy, safety, serviceability, functional obsolescence and special reductions) to obtain a numeric value that is indicative of the bridge's sufficiency to remain in service. The result of this method is a percentage in which 100 is an entirely sufficient bridge and 0 is an entirely deficient bridge.

The following identifies the specific criteria for determining structural deficiency or functional obsolescence:

1. Structurally Deficient. A condition of "4" or less for:

   58) Deck or
   59) Superstructure or
   60) Substructure or
   62) Culvert

   Or, an appraisal of "2" or less for:

   67) Structural Evaluation
   71) Waterway Adequacy

2. Functionally Obsolete. An appraisal of "3" or less for:

   68) Deck Geometry or
   69) Under clearances or
   72) Approach Roadway Alignment

   Or, an appraisal of "3" for:

   67) Structural Evaluation
   71) Waterway Adequacy

Note: Any bridge classified as structurally deficient is excluded from the functionally obsolete category. Bridges shown as built or rehabilitated in the last 10 years are not eligible. However, once a bridge is on the eligibility list, it can stay on for 10 years even if the condition and appraisal ratings fluctuate.

22.2.4 MDT Bridge Management System (PONTIS)

The FHWA requires that all State DOTs develop management systems for bridges. This is to ensure that the planning, design, construction and maintenance will produce an optimum use of highway program resources.

The Bridge Management Unit has implemented a Management System called PONTIS. PONTIS is a network-level Bridge Management System that uses a probabilistic model and a bridge database to predict maintenance and improvement needs and to schedule projects within given budget and policy constraints. PONTIS is a tool for budget analysts and managers to develop annual and long-range maintenance and improvement programs and budgets.

The programming of bridge rehabilitation projects is in part based on recommendations from PONTIS. The program also reflects MDT District review and recommendations.

22.2.5 HBRRP

A major source of funding for bridge rehabilitation projects is the FHWA Highway

Bridge Replacement and Rehabilitation Program (HBRRP). The Program provides funds for eligible bridges located on any public road. Montana's share of HBRRP funds is basically determined by its number of structurally and/or functionally deficient bridges compared to the number nationwide.

In addition to replacement and rehabilitation, HBRRP funds may be used for preventive maintenance on highway bridges. Eligible activities include:

1. sealing or replacing leaking joints,

2. applying deck overlays that will significantly increase the deck service life, and

3. painting structural steel.

HBRRP funds available to non-State highway facilities (i.e., off-system) depends on the Federal provision that no less than 15% and no more than 35% of the funds must be used on public roads that are functionally classified as local roads (urban and rural) or rural minor collectors. The Montana Highway Commission has directed the Department to expend the maximum amount possible on off-system bridges.

22.3 CONDITION SURVEYS AND TESTS

To identify the appropriate scope of bridge rehabilitation work, the designer should select and perform the proper array of condition surveys, tests and analyses. This Section provides Department guidance for the designer.

22.3.1 Bridge Bureau Responsibility

22.3.1.1 General

The Bridge Bureau is responsible for:

- participating in field reviews;
- requesting specific tests to be performed by others (e.g., chloride-content analysis);
- evaluating data collected during the field survey and provided by others;
- determining the appropriate scope of rehabilitation or if replacement is appropriate; and
- providing contract documents.

22.3.1.2 Plan Preparation

In addition, at some time during the design and plan preparation phase, the designer should visit the site (or have the District obtain the information) to visually inspect and verify that the condition and configuration of the bridge match what has been assumed during design. In particular, close attention must be given to joints, guard angles and diaphragms. Determine if details match those shown in the plans and shop drawings. Check for evidence of repair work or revisions not indicated in the plans and shop drawings. It may be necessary to schedule the Snooper through the Bridge Management Section to get close enough to the underside of the bridge to observe and evaluate these components.

Within a year of the anticipated letting, another deck condition survey should be made. This is necessary because deck deterioration is constant and on-going. Because the original deck rehabilitation plan is developed from dated information, it is appropriate to check again to validate (or modify as needed) the proposed plans. This evaluation need not be more extensive than chain dragging the deck and verifying that the guard angles are still secure. Refer to Section 22.3.3.2.2 for a description of the chain drag test.

22.3.2 Selection of Surveys/Tests

The decision on the type and extent of bridge rehabilitation is based on information acquired from condition surveys and tests. The selection of these condition surveys and tests for a proposed project is based on a case-by-case assessment of the specific bridge site. The designer should consider the following factors:

1. age;
2. estimated remaining life (i.e., before bridge replacement is necessary);
3. size;
4. historic significance; and
5. potential investment in bridge rehabilitation.

The following information is normally available and may be requested by the designer if deemed pertinent:

1. original design plans and previous rehabilitation plans;
2. as-built plans;
3. shop drawings;
4. pile driving records;
5. previous surveys;

6. accident records;

7. flood and scour data, if applicable;

8. traffic data;

9. roadway functional classification;

10. bridge inspection reports;

11. structural ratings (sufficiency, operating, inventory); and

12. maintenance work performed to date.

Based on an assessment of the structural factors and the available information, the designer will select those condition surveys and tests which are appropriate for the bridge site conditions.

22.3.3 Bridge Decks

22.3.3.1 General

For the purpose of this Chapter, decks include the structural continuum directly supporting the riding surface, deck joints and their immediate supports, curbs, barriers, approach slabs and utility hardware. The bridge deck and its appurtenances provide the following functions:

1. support and distribution of wheel loads to the primary structural components;

2. protection of the structural components beneath the deck;

3. a smooth riding surface; and

4. safe passageway for vehicular and bicycle/pedestrian traffic (e.g., skid-resistant surface, bridge rails, guardrail-to-bridge-rail transitions).

Any deterioration in these functions warrants investigation and possible remedial action. A bridge deck has a finite service life, which is a function of both adverse and beneficial factors in the deck's environment. The most common cause of concrete bridge deck deterioration is the intrusion of chloride ions from roadway deicing agents into the concrete. The chloride causes formation of corrosive cells on the steel reinforcement, and the corrosion product (rust) induces stresses in the concrete resulting in cracking, delamination and spalling. Chloride ion (salt) penetration is a time-dependent phenomenon. There is no known way to prevent penetration, but it can be decelerated such that the service life of the deck is not less than that of the structure. Chloride penetration is, however, not the only cause of bridge deck deterioration. Other significant problems include:

1. Freeze-Thaw. Results from inadequate air content of the concrete. Freezing of the free water in the concrete causes random, alligator cracking of the concrete and then complete disintegration. There is no known remedy other than replacement.

2. Impact Loading. Results from vehicular kinetic energy released by vertical discontinuities in the riding surface, such as surface roughness, delamination and inadequately set or damaged deck joints. Remedial actions are surface grinding, overlay or replacement of deck concrete and rebuilding deck joints.

3. Abrasion. Normally results from metallic objects, such as chains or studs attached to tires. Remedial actions are surface grinding or overlay.

Certain factors are symptomatic indicators that a bridge deck may have a shorter than expected service life or that it is actually in the final phases of its service life. Some examples are:

1. extensive delamination,
2. exposed reinforcing steel, and
3. spalls.

These symptomatic indicators are generally examined at 2-year intervals by bridge inspectors under the auspices of the NBIS. During these inspections, a subjective numerical rating from 0 to 10 is given to the deck based on the nature and extent of these indicators. See Section 22.2.

The deck can be placed into one of the following categories:

1. Very good decks that need little attention. These are the (8) and (9) rated decks. The application of a sealer is considered to be an effective treatment of decks in this condition range.

2. Decks that are in reasonably good shape and need no substantial repair, but their lives can be extended with a nominal maintenance expenditure. These are the (7) rated decks. Decks in this condition range would most likely need some patching.

3. Decks that need considerable repair, but they are still quite sound and capable of serving adequately for several more years. These are candidates for repair and overlay with some type of non-permeable concrete. These are the (5) and (6) rated decks. The designer would most likely be looking at an overlay for bridge decks in this condition range, depending on the extent of chloride contamination. Very few bridge decks in Montana have ratings less than 5.

4. Decks that are no longer serviceable and will soon need replacement regardless of any remedial action. Significant expenditures of funds are not justified until replacement. However, minor maintenance expenditures could extend the remaining life several years. These are the (3) and (4) rated decks. Decks in these conditions then fall into the "replace deck" category.

Although bridge designers rely heavily on NBIS data to focus attention on decks that may need repair, it is not appropriate to rely solely on NBIS data to determine the Department's deck rehabilitation needs. The NBIS inspections are frequently performed during winter. Because of snow cover and inclement weather, it frequently is not possible for the inspector to perform a thorough visual assessment of the condition of the deck. When a bridge deck rehabilitation project is tentatively identified, the bridge designer should request deck surveys on all bridge decks within the limits of the proposed roadway project. There are at least three good reasons to do this:

1. Identify other marginal decks that may not show up based on NBIS screenings.

2. It provides background information for the assessment of deck deterioration rates and future program needs.

3. Visual inspection alone can not provide enough information to assess deck condition.

Ensuring the safety of the traveling public and meeting public demand for "bare" roads in winter requires the use of salts containing chloride (sodium chloride and magnesium chloride) in deicing compounds. Chloride ions from these sources diffuse through the deck, causing corrosion of the reinforcing steel over time. Expanding rust from the steel leads to deck delaminations and spalls.

When considering a bridge for rehabilitation, the Bridge Bureau requests a number of tests to gather information on the deck's condition. The gathered information allows the designer to determine whether deck rehabilitation or deck replacement would use MDT funds more effectively and, if the choice is rehabilitation, the information allows the determination of the appropriate level of treatment.

The Bridge Bureau requests the following information to perform a deck evaluation:

1. a plot locating existing delaminations, spalls and cracks;

2. measurement of the depth of cover on the top mat of reinforcing steel on a grid pattern;

3. sampling and laboratory analysis to determine the existing levels of chloride contamination;

4. measurement of electrical potential on a grid pattern to locate areas of active corrosion; and

5. deck concrete compressive strength assessed through destructive testing of deck core samples.

Section 22.3.3.2 provides more information on the individual forms of data gathered and their use in determining an appropriate deck treatment.

22.3.3.2 Condition Assessment Tests

22.3.3.2.1 Visual Inspection

Description: A visual inspection of the bridge deck should establish:

1. The approximate extent of cracking, delamination, spalling and joint opening.
2. Evidence of any corrosion.
3. Evidence of pattern cracking, efflorescence or dampness on the deck underside.
4. Rutting of the riding surface and/or ponding of water.
5. Operation of deck joints.
6. Functionality of deck drainage system.
7. Bridge rails and guardrail-to-bridge-rail transitions meeting current Department standards.
8. Deterioration and loss of integrity in wood decks.

Purpose: The visual inspection of the bridge deck will achieve the following:

1. By establishing the approximate extent of cracking, corrosion, delamination and spalling (and by having evidence of other deterioration), one can determine if a more extensive inspection is warranted.
2. The inspector will identify substandard roadside safety appurtenances.

When to Use: All potential deck rehabilitation projects.

Analysis of Data: Pattern cracking, efflorescence or dampness on the deck underside suggest that this portion of the deck is likely to be highly contaminated. In addition, the designer should consider:

1. traffic control,
2. timing of repair,
3. age of structure,
4. average annual daily traffic (AADT),
5. slab depth,
6. structure type,
7. depth of cover to reinforcement,
8. seismic factors, and
9. accident history (e.g., wet-weather accidents).

22.3.3.2.2 Delamination Testing or Sounding

Description: Establishes the presence of delamination, based on audible observation, by chain drag or hammer. Based on the observation that delaminated concrete responds with a "hollow sound" when struck by a metal object. See ASTM D4580 *Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding.*

Purpose: To determine the location and area of delamination.

When to Use: On all concrete deck rehabilitation projects, except where asphalt overlays prevent performance of the test.

Analysis of Data: Based on the extent of the bridge deck spalling, the following will apply:

1. 5% delamination of surface area is a rough guide for considering remedial action.

2. 10% delamination is a rough guide for considering bridge deck replacement.

Quantities are approximate for bid purposes only and should be rounded off to the nearest 5%.

22.3.3.2.3 Half-Cell Method

Description: Copper/copper sulphate half-cell method for the measurement of electrical potential as an indicator of corrosive chemical activity in the concrete. See ASTM C876 *Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.*

Purpose: To determine the level of active corrosion in the bridge deck.

When to Use: On all concrete decks that are being evaluated. Even if a concrete deck has a wearing surface, half-cell readings can be made after areas of the deck are exposed.

Analysis of Data: A voltage potential difference of -0.35 volts or less indicates active corrosion as established by FHWA; more recent work suggests that -0.23 volts is the threshold of corrosion. Less negative readings indicate more active corrosion, while higher negative (smaller in absolute value) readings indicate lower corrosion.

22.3.3.2.4 Coring

Description: 50-mm or 100-mm diameter cylindrical cores are taken. In decks with large amounts of reinforcement, it is difficult to avoid cutting steel if 100-mm diameter cores are used.

Purpose: To establish strength, composition of concrete, crack depth, position of reinforcing steel.

When to Use: On all concrete deck rehabilitation projects when questions exist relating to the compressive strength or soundness of the concrete or if the visual condition of the reinforcement is desired. Also, when compression tests are requested.

Analysis of Data: Less than 50 mm of concrete cover is considered inadequate for corrosion protection. Less than 21 MPa compressive strength of concrete is considered inadequate. If compressive strengths are less than 21 MPa, the designer must obtain a determination from the Bridge Area Engineer whether to proceed with the deck rehabilitation or to proceed with a deck replacement. The choice of core locations can have a significant impact on the findings.

22.3.3.2.5 Chloride Analysis

Description: A chemical analysis of pulverized samples of the bridge deck concrete extracted from the deck or by in-place drilling. Concentrations of water-soluble chlorides are determined using the *Gravimetric Method — Silver Chloride Method* as described in *Scott's Standard Methods of Chemical Analysis*, 6th Edition, March 1962, D. Van Nostrand, publisher.

Purpose: To determine the chloride content profile from the deck surface to a depth of about 75 mm or more.

When to Use: Use on all bridge deck evaluations. Take chloride samples at three to five locations from the driving lane per span from each span 30 m or less in length. Increase the number of samples for longer spans.

Analysis of Data: The "threshold" or minimum level of water-soluble chloride contamination in concrete necessary to corrode reinforcing steel is 0.71 kg/m^3 (1.2 lbs/yd^3) or 0.03% chloride by weight. Chloride concentrations equal to or greater that this value above the top reinforcing mat require the removal of at least enough concrete so that the remaining concrete contamination is below the threshold.

Threshold or greater chloride concentrations at the level of the top reinforcing mat require either 1) hydro-demolition to remove enough concrete

to ensure that the remaining concrete is below the threshold values or 2) deck replacement.

Threshold contamination or worse at or near the level of the bottom mat of reinforcing steel requires deck replacement.

22.3.3.2.6 Pachometer Readings

Description: The pachometer produces a magnetic field in the bridge deck. A disruption in the magnetic field, such as induced by a steel reinforcing bar, is displayed.

Purpose: To determine the size, depth and cover of steel reinforcing bars. These properties can be established to a depth of approximately 70 mm.

When to Use: Pachometer readings are used on virtually all concrete rehabilitation projects to verify reinforcement size and location.

Analysis of Data: Depth readings are taken at each grid point and the data is analyzed. If a deck will be mechanically milled or scarified, a removal depth can be selected that will avoid construction problems caused by milling machines snagging reinforcing steel.

22.3.3.2.7 Skid Test

Description: A test performed with a specially designed skid trailer to measure the available frictional resistance between a tire and the aggregate within the pavement surface.

Purpose: To determine if the Skid Number, which represents the frictional resistance, is sufficiently low to present a potential hazard when the pavement is wet.

When to Use: For a bridge rehabilitation project (e.g., Safety) where the structural evaluation of the bridge deck warrants no remedial action but there is a suspicion that the deck's surface may have inadequate skid resistance, especially if there is an adverse history of wet-weather accidents. This test may be used any time the skid resistance of a bridge deck warrants quantification. The decision to perform a Skid Test should be made in coordination with the MDT Safety Management Section.

Analysis of Data: To be performed by the MDT Safety Management Section.

22.3.3.3 Analysis of Multiple-Test Results

Delaminated areas usually indicate high half-cell and chloride content readings. Expect to obtain at least some degree of conflicting test results. Thus, sampling multiple locations within a traffic lane is important to determine the true state of the deck condition and the extent of active corrosion. Even if unsubstantiated by test results, the designer should assume that at least 1% of the deck area will require full-depth patching when estimating the project cost and determining the project scope.

Engineering judgment should be applied in analyzing test results.

22.3.4 Superstructure

For this Chapter, the superstructure includes all structural components located above the bearings, except decks. For bridges without bearings, such as rigid frames, fixed arches, etc., this includes every visible structural component, except decks. The following briefly describes those condition surveys and tests which may be performed on the superstructure elements to determine the appropriate level of rehabilitation.

22.3.4.1 Visual Inspection

Description: A visual inspection of the superstructure should include an investigation of the following:

1. Surface deterioration, cracking and spalling of concrete.

2. Major loss in concrete components.

3. Evidence of efflorescence.
4. Corrosion of reinforcing steel or prestressing tendons.
5. Loss in exposed reinforcing steel or prestressing tendons.
6. Corrosion of structural metal components.
7. Loss in metal components due to corrosion.
8. Cracking in metal components.
9. Excessive deformation in components.
10. Loosening and loss of rivets or bolts.
11. Deterioration and loss in wood components.
12. Damage due to collision by vehicles, vessels, ice or debris.
13. Leakage through deck joints.
14. Ponding of water on abutment seats.
15. State and functionality of bearings.
16. Distress in pedestals and bearing seats.

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

When to Use: On all bridge rehabilitation projects.

Analysis of Data: As required.

22.3.4.2 Fracture-Critical Members

A fracture-critical member is a metal structural component, typically a superstructure tension or bending member, which would cause collapse of the structure or span if it fails. Fracture-critical structures in Montana have been identified and are noted in the inspection records on file in the Bridge Management Section. The designer must recognize typical fracture-critical details when conducting the Preliminary Field Review because it may affect the scope of bridge rehabilitation. Typical bridges in Montana containing fracture-critical members are listed below:

1. Steel trusses (pins, eye-bars, bottom chords and other tension members).
2. Two-girder steel bridges.
3. Transverse girders (supporting longitudinal beams and girders).
4. Pin-and-hanger connections (located on suspended spans or at transverse girders).

22.3.4.3 Tests for Cracking in Metals

The extent and size of cracks should be established to determine the appropriate remedial action if visual inspection reveals cracking in steel components. The following are the most common test methods used in locating cracks in steel components and measuring their extent and size:

1. Dye-Penetrant Testing. The surface of the steel is cleaned, then painted with a red dye. The dye is wiped off. If a crack is present, the dye penetrates the crack. A white developer is painted on the cleaned steel and any cracks are indicated where the red dye "bleeds" from the crack.
2. Magnetic-Particle Testing. The surface of the steel is cleaned and sprinkled with fine iron filings while a strong magnetic field is induced in the steel. Magnetism is not resisted by the void in the cracks; therefore, the particles form a footprint thereof.
3. Radiographic Testing. This is a highly reliable but cumbersome and expensive test because it requires a medium producing x-rays which penetrate the cracks and mark the film located at the other side. The film provides a permanent record of the x-ray test. Public and operator safety is an issue

when using an x-ray source on an existing bridge.

4. Ultrasonic Testing (UT). Testing devices that use high-frequency sound waves to detect cracks, discontinuities and flaws in materials. The accuracy of UT depends upon the expertise of the individual conducting the test and interpreting the results.

All tests must be conducted by, at a minimum, a Level II ANSI approved technician. For more information, see *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, NCHRP Report 206, June 1979.

22.3.4.4 Fatigue Analysis

Description: Fatigue is defined as crack growth to a size at which fracture is no longer effectively resisted, leading to failure of the component. The crack growth is a function of:

1. crack size;
2. location of crack (i.e., structural detail);
3. energy-absorbing characteristics of metal;
4. temperature; and
5. frequency and level of stress range (transient stresses).

Purpose: To establish type and urgency of remedial action.

When to Use: Where cracks, found by visual inspection, are believed to be caused by fatigue or at fatigue-prone details.

Analysis of Data: Analysis should be performed by a structural engineer, experienced in fatigue-life assessment.

22.3.5 Substructures/Foundations

The substructures of the bridge transfer loads to rock or soil. Substructures include piers, bents and abutments, footings, driven piles and drilled shafts. Substructures including driven piles and drilled shafts are referred to as "deep foundations." Substructures including spread footings are referred to as "shallow foundations." The following briefly describes those condition surveys and tests which may be performed on these elements to determine the appropriate level of rehabilitation.

22.3.5.1 Visual Inspection

Description: A visual inspection of the substructure components should address the following:

1. Surface deterioration, cracking and spalling of concrete.
2. Major loss in concrete components.
3. Evidence of corrosion of reinforcing steel.
4. Loss in exposed reinforcing steel.
5. Deterioration or loss of integrity in wood components.
6. Leakage through joints and cracks.
7. Dysfunctional drainage facilities.
8. Collision damage.
9. Changes in geometry such as settlement, rotation of wing walls, tilt of retaining walls, etc.
10. Seismic vulnerabilities.
11. Accumulation of debris.
12. Erosion of protective covers.

13. Changes in embankment and water channel.

14. Evidence of significant scour.

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

When to Use: On all potential bridge rehabilitation projects.

Analysis of Data: As required.

22.3.5.2 Other Test Methods

Other test methods described in Section 22.3.3 for bridge decks may be used to determine the level and extent of deterioration of concrete substructure components. The test methods described in Section 22.3.4.3 for cracking of metal components may be used for metal substructures.

22.4 BRIDGE REHABILITATION TECHNIQUES

As discussed in Section 22.3, the bridge condition surveys, tests, analyses and reports will indicate the extent of the problems and the objectives of rehabilitation. Section 22.4 presents specific bridge rehabilitation techniques that the designer may employ to address the identified deficiencies. This Section is segregated by structural element (i.e., bridge decks, steel superstructures, concrete superstructures, substructures/foundations and seismic retrofit). For each technique, Section 22.4 presents a brief description.

In addition, where applicable, several typical Department practices are presented which apply to bridge rehabilitation projects. The discussion in Section 22.4 is not intended to be all inclusive, but it provides the designer with a good starting point on the more common bridge rehabilitation techniques used by MDT. On individual projects and for individual applications, the designer is encouraged to review recent highway engineering literature for more information and to consult with the Bridge Area Engineer for assistance in determining an appropriate course of action. See Section 22.7.

22.4.1 Bridge Decks

22.4.1.1 Manual Reference

Chapter Fifteen of the **Montana Structures Manual** provides an in-depth discussion on the design of decks for new bridges. Many of the design and detailing principles provided in the Chapter also apply to deck rehabilitation. Therefore, the designer should review Chapter Fifteen to determine its potential application to the bridge rehabilitation project.

22.4.1.2 Typical Department Practices

The Department has adopted several typical practices for the rehabilitation of bridge decks. These are enumerated in the following:

1. Bridge Deck Overlays. The following summarizes typical Department practices:

   a. Patching. Patching the bridge deck should be considered a temporary measure to provide a reasonably acceptable riding surface until a more permanent solution can be applied.

   b. Latex-Modified Overlay. This is typically applied in conjunction with deck patching. Since the 1970s, the latex-modified overlay has been the most common bridge overlay technique used.

   c. Bituminous Overlay with Sheet Membrane. This method is considered a last resort treatment to extend the deck life until a replacement deck can be programmed. Because of MDT's desire to maintain visual surveillance of the concrete deck surface, the designer must obtain the approval of the Bridge Engineer before proceeding with this option. Breakdown of the membrane underneath the overlay and difficult construction tolerances preclude its further use.

   d. Low-Slump Concrete. This is also generically referred to as the "Iowa Deck." These were dense, low-slump concrete overlays, 50-mm to 60-mm thick, which were specified as an alternative to latex-modified overlays for over 25 years. Because this product has similar characteristics as the latex-modified overlay and is more expensive, it is no longer specified.

   e. Second Overlays. Department policy is not to allow a new overlay to be placed over an existing bridge deck overlay, because it is counterproductive and adds to the dead-weight of the structure.

2. Joints. The Department recognizes that the service life of bridge deck expansion joints is much shorter than that of the bridge, and leaking and faulty joints represent a hazard

for the deck and the main structural components. Therefore, the Department's standard procedure is to eliminate expansion joints as part of the bridge rehabilitation project where practical. Where applicable, the bridge deck rehabilitation should be consistent with the Department's criteria in Section 15.3 on the design of bridge deck expansion joints.

Compression seals (Type BS joints) are not allowed on bridge deck rehabilitation projects, and all such existing joints should be removed during rehabilitation.

3. Minimum Class A or Partial-Depth Patching Quantities. In general, the quantity summaries for bridge rehabilitation projects only include an estimate of the percent of bridge deck patching; the exact amount of patching needed is determined in the field during construction. However, the minimum amount of bridge deck patching shown in the quantities summary should be 5% of the bridge deck area.

22.4.1.3 Rehabilitation Techniques

The following pages present a brief description on those bridge deck rehabilitation techniques that may be considered on Department projects. The designer should review the technique and determine its applicability to the project. The techniques include:

- Technique BD-1 "Deck Repair"
- Technique BD-2 "High Molecular Weight Methacrylate (HMWM)"
- Technique BD-3 "Concrete Overlay"
- Technique BD-4 "Deck Drainage Improvements"
- Technique BD-5 "Joint Elimination"
- Technique BD-6 "Joint Replacement"
- Technique BD-7 "Silane Sealers"
- Technique BD-8 "Membrane with Asphalt Overlay"
- Technique BD-9 "Approach Slabs"
- Technique BD-10 "Introduce Composite Action"
- Technique BD-11 "Wood Deck Replacement"

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-1
Title: Deck Repair

Description:

This rehabilitation technique is used under two distinctly different circumstances. One possible application is to repair isolated popouts or delaminations and restore the driving surface of the deck; another possible application is the repair of unsound areas after scarification or hydrodemolition of a deck that will receive an overlay.

If the intended application is to repair isolated popouts and restore the driving surface, consider the following guidance. The area to be patched is defined by sounding. Boundaries of the area are sawed at least 450 mm outside of the delaminated area to a depth of at least 13 mm. The concrete is then removed. Any exposed reinforcing steel is cleaned. A bonding agent is then applied to the existing concrete surface. Usually, a sand-cement grout or epoxy bonding agent is brushed onto the concrete surface.

Although conventional portland cement concrete is often used, other materials have been developed to permit early opening of the deck to traffic, such as accelerators, and fast-setting cements.It is essential that the manufacturers' specifications for mixing, placing and curing be rigidly followed.

Deck patching alone is usually only moderately successful and should be considered as a stopgap measure to extend the service life of the deck until overlay or replacement is justified.

The designer must prepare a special provision setting forth the work to be done on the specific project.

MDT uses two classifications of deck repair. Partial-depth patching is called Class A patching; full-depth patching is called Class B patching.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-2
Title: High Molecular Weight Methacrylate (HMWM) (Low Viscosity Sealants for Crack Repairs)

Description:

A low-viscosity organic liquid compound is flooded over the deck, and it fills the cracks by gravity and capillary action. Accordingly, the success of this operation depends on the crack size, selection of the appropriate compound, temperature, contamination on the crack walls and the skill of the operator.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-3
Title: Concrete Overlay

Description:

This rehabilitation technique is used for several purposes. Its most common application is for the re-establishment of a riding surface after scarification or hydrodemolition have removed chloride-contaminated concrete. The thickness and impermeability of the overlay reduce the rate of the chloride defusion with a resulting increase in deck life:

1. Deck Preparation. Surface milling or scarification usually removes the top 6 mm of the entire bridge deck surface. Hydrodemolition can be more precisely controlled to remove only the concrete that is unsound.

   Following the clean up from the surface removal operation, areas of unsound concrete are marked for further removal. Removal of the unsound concrete should be performed by either handchipping or hydrodemolition. Jack hammers should not be heavier than nominal 20.5-kg class, and chipping hammers than nominal 6.8-kg class. Hydrodemolition equipment should be calibrated to remove only unsound concrete.

   The removal areas should be thoroughly cleaned to assure the complete bonding of the new concrete to the old concrete.

2. Patching. Cavities left after the concrete removal operation should be patched prior to overlay by either normal portland cement concrete or latex-modified concrete. The cavity should be filled to the level of adjacent concrete.

A latex-modified overlay can be placed when the concrete that has been placed in the cavities has adequately cured. Latex-modified concrete has unique handling and finishing properties and, if the contractor is not familiar with these properties, the possibility exists that an unsatisfactory product will result.

After finishing, the surface is given a burlap drag finish. Transverse grooves will be sawn in the deck after the cure is complete.

The overlay should receive a wet cure for a minimum of 24 hours, followed by 72 hours of dry cure. In lieu of 72 hours of dry cure, the overlaid bridge deck may be opened any time if the compressive strength of the latex-modified concrete exceeds 27.5 MPa. Burlap for wet curing should be placed as soon as the overlay surface supports it without deformation. Approximately one hour after placing the first layer of burlap, a second layer, consisting of wet burlap or polyethylene film, should be placed and secured in position.

MDT has developed a "standard" Special Provision to address this work.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-4
Title: Deck Drainage Improvements

Description:

The most common drainage problems are:

1. deterioration around drainage facilities,
2. an inadequate number of facilities,
3. clogging of facility due to insufficient size and lack of maintenance, and
4. spilling water onto other structural components or the roadway below and/or causing erosion.

Details should ensure positive attachment of the facility to the existing structure and permit proper compaction of the new concrete in the deck.

See Section 15.3.8 for more information on bridge deck drainage.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-5
Title: Joint Elimination

Description:

On many bridges, the deck joint may be eliminated by simply making the concrete deck continuous. This can be achieved by removing sufficient concrete on both sides of the joint to permit adequate lap joints in the longitudinal steel, then form and place the concrete.

The structural implications of joint removal should be investigated:

1. A portion of the deck concrete is removed to permit placement of deck steel.
2. The effects of additional longitudinal movements must be investigated at the remaining joint locations.
3. For integral and semi-integral superstructures, consider the effects of cumulative movements on the substructures.
4. Consider the need for discontinuity in the barriers at the points where the joints are eliminated.
5. If two bearings are used, consider the effects of increased eccentricity of reaction forces on the substructures.

Making decks continuous by eliminating joints generally improves the seismic performance of a bridge. Adequate lap splices and proper development length are required along with sufficient reinforcement to transfer the loads from one span to another. The reinforcement should approximately match the reinforcement in as-built continuous slabs over simply supported beams.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-6
Title: Joint Replacement

Description:

Short of eliminating a joint, a simple replacement of an existing damaged or malfunctioning joint may be part of a bridge rehabilitation project. Joint replacement may be made where joint elimination is not possible due to structural or practical reasons.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-7
Title: Silane Sealers

Description:

One method of preventing the entry of chloride ions into the concrete is sealing its surface. In Montana, the useful life of this sealant is usually no more than three years. However, the minor costs associated with this technique give it a favorable cost-benefit ratio.

MDT maintains an approved list of sealers, which includes information identifying the manufacturer, sealer designation and additional requirements for specific sealers,

The designer must prepare a Special Provision setting forth the work to be done.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-8
Title: Membrane with Asphalt Overlay

Description:

MDT has traditionally discouraged placing asphalt overlays on bridge decks. This is primarily due to the reduced ability to properly inspect bridge decks covered with asphalt. Other problems associated with covered decks include added dead load, which reduces live load capacity, and trapping of moisture in the concrete, further aggravating corrosion of the slab reinforcing steel.

In recent years, the Bridge Bureau has designed and constructed a few deck rehabilitation projects using membrane systems in conjunction with asphalt overlays. These overlays were placed in selected areas on aging decks that were near the end of their useful lives and where replacement of the deck or entire bridge was being considered in the near future.

Where the existing deck surface is spalling or delaminated and traffic control issues demand a quick fix for ride improvement, this system will generally result in a reasonably smooth surface with little expense. Where the existing concrete deck is distorted or out of plane due to poor initial construction or due to settlement, this method has had limited success in providing ride improvement.

To use this overlay system, the Bridge Area Engineer must document the project-specific data justifying its use and obtain the approval of the Bridge Engineer.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-9
Title: Approach Slabs

Description:

Abutments for on-system bridges are typically designed to accommodate a future approach slab. As such, the addition of an approach slab should be considered during bridge rehabilitation if distress of the approach pavement due to settlement or lateral displacement is observed.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-10
Title: Introduce Composite Action

Description:

Introducing composite action between the deck and the supporting beams is a "natural" way to increase the strength of the superstructure. The **LRFD Bridge Design Specifications** encourage the use of composite action where current technology permits. Composite action can be achieved by welded studs.

Composite action considerably improves the strength of the upper flange in positive moment areas.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: BD-11
Title: Wood Deck Replacement

Description:

On older bridges with wood decks (especially trusses), it may be impossible to rehabilitate the bridge with a new replacement concrete deck, because the increased dead load of the concrete deck will reduce the available live-load capacity. In such cases, deteriorated wood decks may be replaced in kind with a glue-laminated or nail-laminated wood deck.

22.4.2 Concrete Components Below the Deck

22.4.2.1 Manual Reference

Chapters Sixteen, Seventeen, Nineteen and Twenty of the **Montana Structures Manual** provide a detailed discussion on the design of concrete components below the decks of new bridges of reinforced concrete and prestressed concrete. Many of the design and detailing principles provided in these Chapters also apply to the rehabilitation of an existing concrete bridge. Therefore, the designer should review those Chapters to determine their potential application to the bridge rehabilitation project.

22.4.2.2 Rehabilitation Techniques

The following pages present a brief discussion on those concrete rehabilitation techniques which may appropriate for rehabilitating concrete portions of superstructures and substructures. These include:

- Technique CC-1 "Remove/Replace Deteriorated Concrete"
- Technique CC-2 "Shotcrete"
- Technique CC-3 "Epoxy Injection"
- Technique CC-4 "Post-Tensioning Tendons — Strengthening

Bridge Rehabilitation Technique
CONCRETE COMPONENTS

Reference Number: CC-1
Title: Remove/Replace Deteriorated Concrete

Description:

A clean, sound surface is required for any repair operation; therefore, all physically unsound concrete, including all delaminations, should be removed.

To prevent removing sound concrete, pneumatic hammers should be restricted to 14 kg for surface operation, and to 7 kg for chipping below steel. Saw-cut the edges of removal areas to a minimum of 13 mm. If the reinforcement bars are rusted, they must be exposed. Loose bars should be tied at each intersection point. Finally, the existing concrete surface and the exposed bars should be blast cleaned.

The remaining concrete should be capable of resisting its weight, any superimposed dead load, live load (if the bridge will be repaired under traffic), formwork, equipment and the plastic concrete. The formwork should resist the plastic concrete without slipping or bulging. Prior to placing concrete, the forms should be cleaned, oiled and wetted.

If the concrete surface is cleaned by high-pressure water blasting, it should be allowed to dry before any epoxy bonding agent or cement paste is applied. The new concrete should be applied before the bonding agent sets.

Bridge Rehabilitation Technique
CONCRETE COMPONENTS

Reference Number: CC-2
Title: Shotcrete

Description:

Instead of placing the new concrete in forms, it may be applied at high velocity by a pump through a hose and nozzle. For this application, the concrete should have a high cement content, low water-cement ratio, and the coarse aggregates replaced by fine aggregates.

Forming thin patches on vertical and overhead surfaces is often difficult as is placing and consolidating thick layers. This method may not be economical for small jobs because of the high mobilization costs.

For small areas, latex-modified concrete or mortar may be employed. Troweling or other finishing should be discouraged because they tend to disturb bonding. Scraping and cutting may be used to remove high points or material that has exceeded the limits of the repair after the concrete has become sufficiently stiff to withstand the pull of the cutting device.

Dimensions are difficult to control with this method, and the finish is often rough. It should not be used on exposed surfaces in urban areas.

The designer must prepare a Special Provision setting forth the work to be done.

For additional information, see the FHWA Workshop Notebook **Rehabilitation of Existing Bridges**, 1984.

Bridge Rehabilitation Technique
CONCRETE COMPONENTS

Reference Number: CC-3
Title: Epoxy Injection

Description:

Epoxy resin injection is commonly used to fill cracks in substructure units. Because the resin is injected under pressure, it is possible to fill nearly all of the cracks. Reinforcing bars are located with a Pachometer and holes are drilled to an appropriate depth into the cracks between reinforcing bars. The crack between the injection ports is sealed with a putty-like epoxy applied to the concrete surface by hand. Injection ports are placed at the holes, and a suitable epoxy system capable of bonding to wet surfaces is injected into the entry hole under pressure until it appears in the exit hole(s). A pumping system, in which the two components of the epoxy are mixed at the injection nozzle, is usually employed.

For selecting the epoxy resin and for the method of application, advice from the suppliers of the resin should be sought.

The designer must prepare a Special Provision setting forth the work to be done.

Bridge Rehabilitation Technique
CONCRETE COMPONENTS

Reference Number: CC-4
Title: Post-Tensioning Tendons — Strengthening

Definition: The addition of post-tensioned tendons to restore the strength of the prestressed concrete beam where original strands or tendons have been damaged. Strengthening by post-tensioning is also applied to non-prestressed concrete beams or hammerhead piers and not only as a result of collision.

Application: Collision of overheight vehicles or equipment with a bridge constructed with prestressed concrete beams may result in breaking off the concrete cover and subsequent damage to or severing of the beam tendons. Exposure to water and salt may also cause damage, particularly where the concrete cover is damaged or cracked. Because the steel tendons determine the load-carrying capacity of the beam, any damage impairs resistance and must be repaired. Transverse cracking of hammerhead piers is a candidate for external longitudinal post-tensioning along the sides of the hammerhead to close the cracks.

Procedure. At a minimum, the following steps apply:

1. Conduct a structural evaluation to determine the extent of the damage.

2. Evaluate the existing diaphragms to ensure their adequacy to support the end anchorage of the tendons.

3. Determine the placement of the temporary load to be applied to existing beams prior to removal and placement of concrete in prestressed concrete beams, if any.

The post-tensioning system should be designed and constructed in accordance with the manufacturer's recommendations. All wedge-type anchorages are susceptible to seating losses; therefore, for short lengths, rolled steel bars are preferred.

Special Note: The designer shall prepare a special provision setting forth the work to be accomplished for completion of this technique on a specific project. This special provision shall be included in the contract documents.

Reference: FHWA Workshop Notebook **Rehabilitation of Existing Bridges**, 1984.

22.4.3 Steel Superstructures

22.4.3.1 Manual Reference

Chapter Eighteen of the **Montana Structures Manual** provides a detailed discussion on the structural design of steel superstructures for new bridges. Many of the design and detailing practices provided in that Chapter also apply to the rehabilitation of an existing steel superstructure. Therefore, the designer should review Chapter Eighteen to determine its potential application to bridge rehabilitation projects.

22.4.3.2 Rehabilitation Techniques

The following pages present a brief discussion on those steel superstructure rehabilitation techniques which may be considered on Department projects. These include:

- Technique SS-1 "Grinding"
- Technique SS-2 "Peening"
- Technique SS-3 "Drilled Holes"
- Technique SS-4 "Bolted Splices"
- Technique SS-5 "Welding"
- Technique SS-6 "Addition of New Stringers — Strengthening"
- Technique SS-7 "Bearings"
- Technique SS-8 "Heat-Straightening"
- Technique SS-9 "Painting"
- Technique SS-10 "Pin and Hanger Rehabilitation"

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-1
Title: Grinding

Description:

If the penetration of surface cracks is small, the cracked material can be removed by selective grinding without substantial loss in structural material. Grinding should preferably be performed parallel to the principal tensile stresses, and surface striations should carefully be removed because they may initiate future cracking.

Grinding can be used when beams are nicked while sawing off old decks.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-2
Title: Peening

Description:

Peening is an inelastic reshaping of the steel at the surface location of cracks, or of potential cracks, by using a mechanical hammer. This procedure not only smooths and shapes the transition between weld and parent metal, it also introduces compressive residual stresses that inhibit the cracking. Peening is most commonly used at the ends of cover plates to reduce fatigue potential.

A new computer-controlled peening process utilizing high-speed peening called ultrasonic peening has been introduced, which removes the dependency of the quality of mechanical-hammer peening on the operator's proficiency. This process promises weld enhancement for unavoidable poor fatigue resistance details such as terminations of longitudinal stiffeners.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-3
Title: Drilled Holes

Description:

At the sharp tip of a crack, the tensile stress exceeds the ultimate strength of the metal, causing rapid progression if the crack size attains a critical level. The purpose of drilled holes is to blunt the sharp crack tip. The location of the tip should therefore be established by one of the crack detection methods provided in Section 22.3.4.3. Missing the tip renders this process useless.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-4
Title: Bolted Splices

Description:

Where rivets or bolts in a connection are replaced, or where a new connection is made as part of the rehabilitation effort, the strength of the connection should not be less than 75% of the capacity or the average of the resistance of and the factored force effect in the adjoining components. Almost exclusively, the connections are made with high-strength bolts (ASTM A325). The connection must be designed by a structural engineer.

This method can also be used to span a cracked flange or web, provided that such connection is designed to replace the tension part of the element or component.

The preferred method of tightening bolts is by direct tension indicators; however, the designer must be aware that, if only a few bolts will be installed, an alternative method to control bolt tension such as calibrated torque wrenches or the "turn-of-nut" method, are acceptable. Regardless of the method used, all the bolts in the group are brought into a "snug-tight" condition and, then, the bolts are individually tightened to the specified tension.

For drilling holes, washers, tightening bolts and ensuring adequate pretensioning, the designer should refer to Section 556 of the **MDT Standard Specifications**.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-5
Title: Welding

Description:

It is common practice to use welding for shop fabrication of steel members and for welding pieces in preparation for rehabilitation work. Field welding is often difficult to perform properly in high-stressed areas, and individuals with the necessary skill and physical ability are required. The proper inspection of field welds is equally difficult. A shop weld is preferred to a field weld. All welding, whether in the shop or in the field, must be performed by a certified welder using welding processes and materials as approved on their certification card.

Field welding should only be allowed on secondary members, for temporary repairs, or in areas where analysis shows minimal fatigue stress potentials.

See Section 556 of the **MDT Standard Specifications** for additional specifications for welding of steel.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-6
Title: Addition of New Stringers — Strengthening

Description:

If the deck is removed, a new set of stringers added to the existing bridge is one alternative to strengthen the superstructure. To ensure proper distribution of live load, rigidity of the new stringers should be close to that of the existing ones.

The old stringers may also need rehabilitation, in which case, their removal may be considered as both a structurally and economically more proper alternative. The presence of lead paint may make replacement more economically feasible. Using modern deck designs and composite action, continuous stringers with a large spacing should be explored as an alternative.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-7
Title: Bearings

Description:

Often, the existing bearings may only need cleaning or repositioning. Extensive deterioration, or frozen bearings, may indicate that the design should be modified. A variety of elastomeric devices may be substituted for sliding and roller bearing assemblies. If the reason for deterioration is a leak in the deck joint, it should be sealed.

Rocker bearings and elastomeric bearings should not be mixed on the same pier/bent, due to differences in movement.

If the bearing is seriously dislocated, its anchor bolts badly bent or broken, or the concrete seat or pedestal is structurally cracked, the bridge may have a system-wide problem usually caused by temperature or settlement, and should be so investigated.

The bearing design may require alteration if warranted by seismic effects.

See Section 19.3 of the **Montana Structures Manual** for more information on bearings.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-8
Title: Heat Straightening

Description:

This technique is restricted to hot-rolled steels. Steels deriving their strength from cold drawing or rolling tend to weaken when heated. The basic idea of heat straightening is that the steel, when heated to an appropriate temperature (usually cherry color), loses some of its elasticity and deforms plastically. This process rids the steel of built-up stresses. While at an elevated temperature, the steel can also be hot worked and forced into a desirable shape or straightness without loss of ductility. Special care should be exercised not to overheat the steel; accordingly, this technique should be implemented by those having experience with this process. Note also that the heating temporarily reduces the resistance of the structure. Measures such as vehicular restriction, temporary support, temporary post-tensioning, etc., may be applied as appropriate.

Bridge Rehabilitation Technique
BRIDGE DECKS

Reference Number: SS-9
Title: Painting

Description:

Technically, bridge painting is maintenance work and not rehabilitation work but, frequently, during project development painting is discussed in conjunction with rehabilitation work on steel structures. In general, bridge painting is not economical but, in some circumstances, it may be warranted on a specific project. When considering bridge painting options, three scenarios present themselves. These are:

1. full removal of existing paint and repainting,
2. a complete recoat over the top of the existing paint (overcoat), and
3. touch-up painting.

The single driving factor in all discussions on painting bridges is that virtually all paint applied to bridges prior to 1977 contained lead. To remove existing paint, the current state of practice is abrasive blast removal, full enclosure, environmental and worker monitoring. The price for all this work approaches, and at times exceeds, the cost of replacing the existing steel bridge members with weathering steel.

The paint industry has developed products that can be successfully applied over existing paints and marginally prepared surfaces. An overcoat may be an economic alternative to full removal and repainting where a uniform appearance for the structural members is desired at the conclusion of the rehabilitation, but the problems associated with lead-based paints are not solved, merely deferred until a subsequent rehabilitation or structure replacement. Touch-up painting neither gives a uniform appearance nor solves the long-term lead problem. Touch-up painting may be appropriate in localized zones where corrosion could cause section loss.

Careful consideration must be given to the proper selection of paint for an overcoat. An improperly specified or improperly applied overcoat can cause failure of the original paint that was performing satisfactorily. Close attention must be given to the manufacturer's literature on any paint's service environment and recommended application environment. Proper surface preparation, application and field inspection are 80% of the challenge in applying paint.

See Section 612 of the **MDT Standard Specifications** for additional specifications on painting.

Bridge Rehabilitation Technique
STEEL SUPERSTRUCTURES

Reference Number: SS-10
Title: Pin and Hanger Rehabilitation

Description:

Pin and hanger details were originally used to facilitate the analysis of bridges by providing pins in otherwise continuous bridges. Their use today is not necessary due to modern computer-based structural analysis. These details are particularly susceptible to corrosion. Corrosion can result in the initiation of fatigue crackings in the hangers due to frozen pins and the unseating of the hangers on the pins due to misalignment from the corrosion product. The infamous collapse of one span of the Mianus River Bridge on I-95 in Connecticut was the result of corrosion of a pin and hanger detail.

Three solutions are possible for pin and hanger details:

1. Unlock frozen pins and hangers. The pin and hanger detail can be disassembled after providing alternative support to the suspended girder. Then, the various components of the detail can be cleaned of rust and dirt or replaced before re-assembly.

2. Provide a catch girder. As a safeguard against failure, especially for fracture-critical girders, an alternative permanent support system can be fabricated to "catch" the suspended girder ends if the pin and hanger detail fails. Such a structure must be temporarily provided to perform the unlocking of frozen details discussed above.

3. Eliminate the pin and hanger detail. If the girder sections allow, a bolted splice of the web and flanges can be fabricated to replace the pin and hanger. A structural analysis of the resulting continuous structure must verify that the resulting loads do not exceed the resistance of the existing girder section.

22.4.4 Substructures/Foundations

22.4.4.1 Manual Reference

Chapters Nineteen and Twenty of the **Montana Structures Manual** provide a detailed discussion on the structural design of substructures and foundations for new bridges. Many of the design and detailing principles provided in these chapters also apply to the rehabilitation of the substructures and/or foundations of an existing bridge. Therefore, the designer should review Chapters Nineteen and Twenty to determine their potential application to the bridge rehabilitation project.

22.4.4.2 Foundations for Bridge Widening

When a bridge will be widened, it is usually prudent to order cores to determine soil engineering properties for the design of additional substructure elements. The bridge designer should send a copy of the existing core logs (if they exist) to the Geotechnical Section. If the new core logs conflict with the old cores, then additional coring may be required.

If the MDT has cores for a bridge but they are not included in the contract documents, then the bridge designer should note in the construction plans that the cores are available for review in the Bridge Bureau.

22.4.4.3 Rehabilitation Techniques

The following pages present a brief discussion on those substructure and foundation rehabilitation techniques which may be considered on Department projects. These include:

- Technique SF-1 "Enlarge Footings"
- Technique SF-2 "Riprap"
- Technique SF-3 "Wing Wall Repair"
- Technique SF-4 "Drainage Improvements"
- Technique SF-5 "Grout Bag Underpinning"
- Technique SF-6 "Pile and Pier Section Loss Repair"

Bridge Rehabilitation Technique
SUBSTRUCTURES/FOUNDATIONS

Reference Number: SF-1
Title: Enlarge Footings

Description:

The most common reasons for enlarging the footings are:

- to widen the structure, or
- excessive settlement, or
- inadequate strength, or
- scour.

The method of rehabilitation is usually one of:

- enlargement of spread footing,
- enlargement of spread footing with piles, or
- enlargement of pile cap with additional piles.

Enlarging an existing spread footing:

1. The preferred alternative is to consult with MDT's Geotechnical Section for appropriate soils information.
2. Where a scour condition exists (spread footing in a stream), extend footings using piles. Designer should consult with MDT's Geotechnical Section for appropriate soils information. Design piles to carry all loads, and do not assume any contribution to the capacity by the footing itself acting as a spread footing.

Enlarging an existing pile-supported footing:

1. Extend the footing with additional piles similar in capacity to the original piles. Check the pile driving records of existing structure.
2. Overhead clearances from beams, decks and cantilever caps should be checked when locating new piles.

For forming, placement of steel, pouring and curing concrete, the same criteria apply as for new construction.

Bridge Rehabilitation Technique
SUBSTRUCTURES/FOUNDATIONS

Reference Number: SF-2
Title: Riprap

Description:

The stability of streambeds and banks is largely a function of water velocity, the size of the material in the streambeds and the size of material and presence or absence of vegetative cover on the banks. The energy of the moving water is a function of the water depth and water velocity. For a given water depth and velocity, if the material size exceeds critical dimensions, scour will not likely occur.

Artificially placed protective material is most usually natural stone that is specifically quarried to be angular for riprap applications, but it can be specially made concrete shapes. For steeper embankments, galvanized, gravel-filled, wire mesh envelopes called gabions can be an option.

The MDT Hydraulics Section will recommend the need for riprap and design its application.

Bridge Rehabilitation Technique
SUBSTRUCTURES/FOUNDATIONS

Reference Number: SF-3
Title: Wingwall Repair

Description:

In many old concrete abutments, the wingwalls tend to break-off and to separate from the main body due to earth-pressure and differential settlement. If the opening has been stable, the do-nothing option may be the best policy. If not stable, the wings should be removed and completely rebuilt. Footings for the new walls should be at the same level as that of the main body.

Bridge Rehabilitation Technique
SUBSTRUCTURES/FOUNDATIONS

Reference Number: SF-4
Title: Drainage Improvements

Description:

Water is a primary cause of instability of fills and embankments. As the water content of a fill behind a retaining structure increases, lateral pressure on the structure is amplified.

If the fill contains excessive amounts of silt or clay, it should be internally drained. This can be achieved either by perforated plastic pipes or by french drains. The latter is a deep trough, the bottom of which is filled with crushed stone or riverbed gravel of equal size. The gravel is covered with a plastic sheet to prevent intrusion of the fill above. Both systems should have exits to ditches permitting unimpaired gravity flow.

Water retention behind retaining structures, such as abutments and walls, is caused either by non-existing or undersized drainage pipes or by clogging thereof. New weep holes of adequate size can be drilled into the concrete if so required. Clogged holes should be thoroughly cleaned.

To prevent future clogging, the entry side of the holes should be provided with a filter and/or a lump of crushed stone or gravel, covered with perforated construction fabric.

Drainage improvement measures that should be considered for preventing erosion of the embankment surfaces at the corners of a structure caused by surface runoff include sodded flumes, erosion control mats, riprap drainage turnouts and curb inlets with piping.

Bridge Rehabilitation Technique
SUBSTRUCTURES/FOUNDATIONS

Reference Number: SF-5
Title: Grout Bag Underpinning

Description:

Scour may cause excessive settlement or tilting of spread footings. Grout-filled bags offer a reasonably simple and economical method of rehabilitation. The construction procedure is as follows:

1. Remove boulders that protrude under footing.
2. Install preformed grout bags and fill with pressurized concrete to mold to and completely fill cavity under the pier.
3. Place grout bags around the periphery of the pier to increase footing size and depth, thereby reducing further potential for undermining.
4. Install horizontal and vertical reinforcement through the grout bags.
5. Drill and grout dowels on 1.0-m centers into the existing seal or footing to anchor new work to old.
6. After jacking and blocking the superstructure, build new seats or pedestals and install the bearings.

Bridge Rehabilitation Technique
SUBSTRUCTURES/FOUNDATIONS

Reference Number: SF-6
Title: Pile and Pier Section Loss Repair

Description:

For steel piles, the following applies to section losses:

1. Small Loss. The restoration of the section of piles that experience a small loss of section associated with "normal" rusting is usually not warranted.

2. Medium Loss. When rusting has reduced the section of the pile such that it becomes a structural concern, the missing cross section is rebuilt by adding plates to the flanges and/or web as appropriate by either welding or bolting.

3. Extensive Loss. When the pile has deteriorated such that there is not enough sound remaining material for the section to be rebuilt, a new pile is installed; the damaged pile may or may not be removed.

For wood piles, section losses may be repaired by:

1. partial replacement,

2. epoxy injection, and/or

3. jacketing.

More information on wood piles can be found in "Timber Bridges – Design, Construction, Inspection and Maintenance" by M. A. Ritter, United States Department of Agriculture, Forest Service, EM 7700-8, June 1990, Chapter 14.

For concrete piles and piers, section loss may be repaired by removing all deteriorated material, constructing a formwork for a jacket, placing a reinforcing steel cage of appropriate size in the formwork and filling it with compacted concrete. The technique has extensive literature on its application.

22.4.5 Seismic Retrofit

22.4.5.1 Responsibility

MDT has developed a program to evaluate the existing bridges on the State highway system. Based on MDT warrants, the Seismic Unit reviews existing bridges for seismic vulnerability and designs the appropriate seismic retrofit on a priority basis.

22.4.5.2 Seismic Evaluation

Earthquakes cause what is best described as a shaking of the entire bridge structure. The ability to predict the forces developed by this motion is limited by the complexity of predicting the acceleration and displacements of the underlying earth material and the response of the structure. The motion can generally be described as independent rotation, in any direction, of each bridge abutment or pier, in or out of phase with each other, combined with sudden vertical displacements. Ground between piers can distort elastically and in some cases rupture or liquefy.

The bridge failures induced by the motions of the abutments and piers stem from two major inadequacies of many existing bridge designs — the lack of adequate connections between segments of a bridge and inadequately reinforced columns. Other deficiencies include inadequately reinforced footing and bent cap concrete and inadequate design force levels considering the likelihood of earthquakes at the location.

Fortunately, tying the segments of an existing bridge together is an effective means of preventing the most prevalent failure mode — spans falling off the bearings, abutments or piers. It is also the least expensive of the inadequacies to correct. Bridges with single-column bents are particularly vulnerable where segments are not connected.

Columns inadequately reinforced, because of too few and improperly detailed ties and spirals or short-lapped splices, generally do not sufficiently confine the concrete. This is particularly critical in single-column bents.

Determining the retrofit technique to use involves these considerations:

1. mode of failure anticipated,
2. influence on other parts of the bridge under seismic and normal loadings,
3. interference with traffic flow, and
4. cost of fabrication and installation.

Some retrofit procedures are designed to correct inadequacies of bridges related to earthquake resistance. The procedures may be categorized by the function the retrofit serves, including:

1. restraining uplift,
2. restraining longitudinal motion,
3. restraining hinges,
4. widening bearings,
5. strengthening columns, and
6. restraining transverse motion.

22.4.5.3 Application

Most of Montana's bridges are in Zone 1. The performance of a seismic evaluation on these existing bridges will be made on a case-by-case basis considering, for example:

1. the scope of the rehabilitation work (i.e., for more extensive rehabilitation work, a seismic evaluation may be appropriate); and
2. the importance of the structure (i.e., for major structures, a seismic evaluation may be appropriate even if the proposed scope of work is limited).

For the rehabilitation of existing bridges within the Montana Districts of Missoula and Butte, the designer is required to perform a seismic evaluation of the structure when major rehabilitation (i.e., deck replacement or superstructure widening) is anticipated.

22.4.5.4 Typical Department Practices

The following summarizes typical Department practices for the seismic retrofitting of existing bridges:

1. General. Bridges that are selected for seismic retrofitting shall be investigated for the same basic criteria that are required for all new bridges, including minimum support length and minimum bearing force demands. Bridge failures have occurred at relatively low levels of ground motion. It is clear, therefore, that MDT's systematic effort to identify seismically deficient bridges is warranted. Specific details for seismic retrofitting may be found in **Seismic Design and Retrofit Manual for Highway Bridges**, FHWA, 1995.

2. Minor. Minor seismic retrofit will usually be limited to seismic restrainers, dynamic isolation bearings and widening of beam seats. For the most part, it will be limited to work at or above the beam seats. The cost of minor retrofits should generally not exceed 25% of the cost of a new, seismically designed structure.

3. Major. Major seismic retrofit includes such items as strengthening columns, piers, bent caps, etc. It will generally include work below the level of the beam seats and may include work requiring cofferdams. The cost of major retrofits should generally not exceed 50% of the cost of a new, structure seismically designed structure.

4. Steel Rocker Bearings. For bridges within the Missoula and Butte Districts, the retrofitting measures shall include modification or elimination of existing steel rocker bearings. Major reconstruction projects in Zone 1 may also be good candidates for the elimination of existing steel rocker bearings, which will be decided on a case-by-case basis.

22.4.5.5 Seismic Retrofit Techniques

The following pages present a brief discussion on those seismic retrofit techniques which may be considered on Department projects. These include:

- Technique SR-1 "Techniques for Increasing Seismic Resistance of Columns"
- Technique SR-2 "Seat Width Extension"
- Technique SR-3 "Structural Continuity"
- Technique SR-4 "Restrainers and Ties"
- Technique SR-5 "Bearing Replacement"
- Technique SR-6 "Seismic Isolation Bearings"
- Technique SR-7 "Integral Abutments"

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-1
Title: Techniques for Increasing Seismic Resistance of Columns

Description:

The following techniques may be used to increase the seismic resistance of columns:

1. Steel Jacket. A solid-steel shell may be placed around the column with a small space which is pressure grouted for a perfect fit.

2. Increase Flexural Reinforcement. If circumstances warrant, the flexural reinforcement may be increased. The vertical bars are located in a concrete jacket which is shear connected to the column by drilled and grouted dowels. This also increases the rigidity of the column, potentially rendering it counterproductive.

3. Infill Shear Wall. A concrete shear wall can be added between the individual columns of the bent. If the existing footing is not continuous, it should be made so. The wall should be connected to the columns by drilled and grouted dowels. This method substantially changes the seismic-response characteristics of the structure, requiring a complete re-analysis.

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-2
Title: Seat Width Extension

Description:

Seat width extensions allow larger relative displacements to occur between the superstructure and substructure before support is lost and the span collapses. The extensions are likely to be exposed to large impact forces due to the dropping span; therefore, they should either be directly supported by the footing or be adequately anchored to the cap. Provisions in the **LRFD Bridge Design Specifications,** relative to the design of seat widths, should be followed as practical.

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-3
Title: Structural Continuity

Description:

Superstructures have often been constructed without longitudinal continuity. Deck joints, beam ends, bearings, bearing seats and piers are potential sources of seismic problems. Structurally unconnected units of the superstructure tend to respond to seismic excitation differently, resulting in the dropping of the bearings or sliding off the substructure.

In older structures, shrinkage, creep and settlement have already occurred, and only the effects of temperature need be considered. The structural behavior of a bridge made continuous is fundamentally different from the non-continuous one, and it should be re-analyzed from every relevant perspective as if it were a new structure. Continuity for seismic purposes can often be attained by making the deck continuous.

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-4
Title: Restrainers and Ties

Description:

In general, restrainers are add-on structural devices which do not participate in resisting other than seismic force effects. Mostly, these components are made of steel, they should be designed to remain elastic during seismic action, and special care should be exercised to protect them against corrosion.

There are three types of restrainers — longitudinal, transverse and vertical. The purpose of the two former ones is to prevent unseating the superstructure. The objective of the third one is to preclude secondary dynamic (impact) forces that may result from the vertical separation of the superstructure.

The restraint devices should be compatible with the geometry, strength and detailing of the existing structure. The designer may need to create new devices if those reported in the literature are not suitable.

Ties are restrainers that connect only components of the superstructure together. They are activated only by seismic excitation.

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-5
Title: Bearing Replacement

Description:

Damaged or malfunctioning bearings can fail during an earthquake. In addition, steel rocker and roller bearings perform poorly for obvious reasons. One option is to replace these bearings with prefabricated steel-reinforced elastomeric bearings. To maintain the existing beam elevation, either a steel assembly is inserted between the beam and the elastomeric bearing, or the elastomeric bearing is seated on a new concrete pedestal. Construction of new concrete pedestals may create significant additional traffic control costs. Existing anchor bolts may assist in resisting shear between the pedestal and the pier. In both cases, the beam should be positively connected to the substructure by bolts, either directly or indirectly.

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-6
Title: Seismic Isolation Bearings

Description:

There is a broad variety of patented seismic isolation bearings which are commercially available. They permit either rotation or translation or both. They have special characteristics by which the dynamic response of the bridge is altered, and some of the seismic energy is dissipated. The primary change in structural response is a substantial increase in the period of the structure's fundamental mode of vibration. The **LRFD Bridge Design Specifications** determine the equivalent lateral static design force as a function of this period. The devices are designed to perform elastically in response to normal service conditions and loads.

Accordingly, seismic isolation bearings normally contain an elastomeric element. The inelastic element is usually either a lead core or a viscous liquid damper whose resistance is a function of the velocity of load application. They are effective for seismic loads due to their high velocity. The liquid dampers are prone to leakage, thus requiring back-up safety devices.

When considering the use of seismic isolation bearings, see Section 19.3 for more information.

Bridge Rehabilitation Technique
SEISMIC RETROFIT

Reference Number: SR-7
Title: Integral Abutments

Description:

One plausible method to provide continuity between superstructures and substructures is the integral abutment. Minimum design requirements for integral abutments are provided in Section 19.1. Integral abutments are only feasible if pile supported and if the arrangement of piles permits the longitudinal temperature movement of the bridge. Cut off existing battered piling below grade.

22.5 BRIDGE WIDENING

22.5.1 Introduction

A bridge widening can present a multitude of problems during the planning and design stages, during construction and throughout its service life. Special attention is required in both the overall design and details of the widening to minimize construction and maintenance problems.

Section 22.5 presents Department guidelines for widening existing bridges. The following briefly summarizes the basic objectives in bridge widening:

1. Match the structural components of the existing structure, including splice locations.

2. Match the existing bearing types in terms of fixity.

3. Do not perpetuate fatigue-prone details.

22.5.2 Existing Structures with Substandard Capacity

An existing structure may have been originally designed for either live loads or seismic loads less than those currently used for new bridges. If such a structure becomes a candidate for widening, the MDT Bridge Management Unit should be consulted on the condition of the existing structure. A rating of the existing bridge must be made to quantify the capacity of the existing bridge. Based on this information, the designer will determine whether the existing structure should be strengthened to the same load-carrying capacity as the widening. For the evaluation, the following should be considered, if appropriate:

1. cost of strengthening existing structure;

2. physical condition, operating characteristics and remaining service life of the structure;

3. seismic resistance of structure;

4. other site-specific conditions;

5. only structure on route that restricts permit loading;

6. width of widening; and

7. traffic accommodation during construction.

22.5.3 Girder Type Selection

In selecting the type of girder for a structure widening, the widened portion of the structure should be a construction type and material type consistent with the existing structure. An exception to this is for an existing conventionally reinforced concrete girder structure. It is preferable to use prestressed concrete I-beams for the widened portion.

22.5.4 Existing Bridge-Deck Assessment

The rehabilitation or replacement of the existing bridge deck shall be considered after an assessment of the existing deck, as discussed in Section 22.3.3.

22.5.5 Epoxy-Coated Bars in Widenings

Epoxy-coated steel reinforcing bars shall be placed in a deck widening if the existing deck contains epoxy-coated bars, and "black" bars shall be used if the existing deck was constructed with "black" bars.

22.5.6 Bridge Deck Longitudinal Joints

Past performance indicates that longitudinal expansion joints in bridge decks between a bridge widening and the existing bridge have been a continuous source of bridge maintenance problems. Therefore, as a general policy, no longitudinal expansion joints should be employed.

Experience has shown that a positive attachment of the widened and original decks by lapping

reinforcing steel provides a better riding deck, usually presents a better appearance and reduces maintenance problems. A positive attachment of the old and the new decks shall be made for the entire length of the structure.

In some cases, it may be desirable to use a type of anchorage system other than lapping reinforcing steel. If the bridge widening exceeds the Department's geometric design criteria for clear roadway width, lapped reinforcing steel may be more expensive than other options because of the need to provide adequate bond length.

The following recommendations should be considered when widening an existing beam/girder and deck-type structure:

1. Structures with large overhangs should be widened by removing the concrete from the overhang to a width sufficient to develop adequate bond length for lapping the original transverse deck reinforcing to that of the widening.
2. Structures with small overhangs, where removal of the overhang will not provide sufficient bond length, should be either doweled to the widening or have transverse reinforcing exposed and extended by mechanical lap splice.
3. Structures with no overhangs should be attached by doweling the existing structure to the widening. Double row patterns for the dowels are preferred over a single row. Benching into the existing exterior girder as a means of support has proven to be unsatisfactory and should be avoided.

22.5.7 Effects of Dead Load Deflection

It is recommended that, where the dead load deflection exceeds 6 mm, the widening should be allowed to deflect and a closure pour considered to complete the attachment to the existing structure. A closure pour serves two useful purposes: It defers final connection to the existing structure until after the deflection from the deck slab weight has occurred; and it provides the width needed to make a smooth transition between differences in final grades that result from design or construction imperfections.

For the effects of dead load deflection, two groups of superstructure types can be distinguished — precast concrete beam or steel beam construction, where the largest percentage of deflection occurs when the deck concrete is placed and, for cast-in-place construction (e.g., reinforced concrete slab bridges), where the deflection occurs after the falsework is released.

In the first group, dead load deflection after placing the deck is usually insignificant but, in cast-in-place structures, the dead load deflection continues for a lengthy time after the falsework is released. In conventionally reinforced concrete structures, approximately ⅔ to ¾ of the total deflection occurs over a four-year period after the falsework is released due to shrinkage and creep. A theoretical analysis of differential deflection that occurs between the new and existing structures after closure will usually demonstrate that it is difficult to design for this condition. Past performance indicates, however, that theoretical overstress in the connection reinforcing has not resulted in maintenance problems, and it is generally assumed that some of the additional load is distributed to the original structure with no difficulty or its effects are dissipated by inelastic relaxation. Good engineering practice dictates that the closure width should relate to the amount of dead load deflection that is expected to occur after the closure is placed. A minimum closure width of 500 mm is recommended.

At the present time, MDT is satisfied with the performance of its bridge decks that are widened without the use of deck closure pours. This satisfactory performance also applies to deck replacements that are poured in two phases while maintaining traffic and without the use of deck closure pours. Consequently, deck widening and phased deck replacement projects normally do not require deck closure pours unless the designer recommends otherwise. An example of when a closure pour may be

warranted is for two-span steel beam/girder structures where uplift could occur.

22.5.8 Vehicular Vibration During Construction

All structures deflect when subjected to live loading, and many bridge widenings are constructed with traffic on the existing structure. Fresh concrete in the deck is subjected to deflections and vibrations caused by traffic. Studies such as NCHRP 86 *Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs* have shown that:

1. good-quality reinforced concrete is not adversely affected by jarring and vibrations of low frequency and amplitude during the period of setting and early strength development;

2. traffic-induced vibrations do not cause relative movement between fresh concrete and embedded reinforcement; and

3. investigations of the condition of widened bridges have shown the performance of attached widenings, with and without the use of a closure pour, to be satisfactory.

22.5.9 Substructure

An existing structure will ordinarily not be subjected to settlement of its footings by the time the widening is completed. Pile capacities of existing structures should be investigated if additional loads will be imposed on them by the widening. It is possible for newly constructed footings under a widened portion of a structure to settle. The new substructure should be tied to the existing substructure to prevent differential foundation settlements. If the new substructure is not tied to the existing substructure, suitable provisions should be made to prevent possible damage where such movements are anticipated.

22.5.10 Design Criteria (Historical Background)

22.5.10.1 AASHTO Standards

It is not normally warranted to modify the existing structure solely because it was designed to AASHTO Specifications prior to the adoption of the **LRFD Bridge Design Specifications** and its latest interim changes.

When preparing plans to modify existing structures, it is often necessary to know the live load and stress criteria used in the original design. Since approximately 1927, with few exceptions, structures on the Montana highway system have been designed for loads and stresses specified by AASHTO.

The designer should be aware of the historical perspective of design criteria, such as live loads, allowable stresses, etc., when analyzing a rehabilitated structure. For accurate and complete information on specific structures, see the General Notes of as-built plans, old standard drawings and special provisions, and the appropriate editions of the AASHTO Specifications.

22.5.10.2 Materials

Figure 22.5A presents the historical properties of materials from the **Montana Standard Specifications for Road and Bridge Construction** since 1946.

22.5.10.3 Rolled Steel Beams

Throughout the years, modifications to rolled steel beam sections have occurred. Designers should refer to the construction-year AISC steel tables for rolled beam properties and other data.

Material	1946	1962	1970	1976	1987	1994
"A" Concrete (f'_c)	2.4 ksi	2.4 ksi	2.4 ksi	2.4 ksi	2.4 ksi	2.4 ksi
"AD" Concrete (f'_c)	3.0 ksi	3.0 ksi	3.0 ksi	3.0 ksi	3.0 ksi	3.0 ksi
"AS" Concrete (f'_c)	NA	NA	2.4 ksi	2.4 ksi	2.4 ksi	2.4 ksi
"BD" Concrete (f'_c)	NA	NA	NA	3.5 ksi	3.5 ksi	3.5 ksi
"DD" Concrete (f'_c)	NA	3.0 ksi	3.0 ksi	3.0 ksi	3.0 ksi	3.0 ksi
"S" Concrete (f'_c)	2.4 ksi	2.4 ksi	NA	NA	NA	NA
Prestressed Beams (f'_c)	NA	5.0 ksi	5.0 ksi	5.0 ksi	5.0 ksi	5.0 ksi
Reinforcement Steel (f'_c)	A15 Int Gr 40 ksi	A15 Int Gr 40 ksi	A-615-68 Gr 40 40 ksi	A-615 Gr 60 60 ksi	A-615 Gr 40 or Gr 60	A-615 Gr 40 or Gr 60
Structural Steel (f_y)	ASTM 47 33 ksi	ASTM A36 36 ksi	ASTM A36 36 ksi	ASTM A36 36 ksi	ASTM A36 36 ksi	ASTM A36 36 ksi

Notes:

1. *Consider increasing typical concrete strengths from the table by 50% for girders and beams to account for conservative batching practices and strength gains from aging. Do not assume strength gain for decks. (See Priestley, Seible & Calvi* ***Seismic Design and Retrofit of Bridges****, pg 546).*
2. 1 ksi = 6.894757 MPa.

MATERIAL PROPERTY VALUES FROM MONTANA STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION

Figure 22.5A

22.6 OTHER BRIDGE REHABILITATION PROJECT ISSUES

A rehabilitation project for an existing bridge requires the consideration of several issues other than the structural design rehabilitation. These include:

1. project development;
2. project reports;
3. plan preparation conventions;
4. geometric design issues;
5. roadside safety issues;
6. maintenance and protection of traffic through construction zones; and
7. other project elements (e.g., hydraulics, geotechnical, environmental procedures, permits, right-of-way).

These topics are discussed elsewhere in the **Montana Structures Manual**. Section 22.6 identifies these references for bridge rehabilitation projects and, where appropriate, provides additional information.

22.6.1 Project Development

Chapter Two presents detailed project development networks for major bridge rehabilitation projects and bridge deck rehabilitation projects. Reference these networks to identify the typical level of effort commensurate with the Project Scope of Work for a given engineering/procedural function (e.g., geotechnical, permits).

22.6.2 Project Reports

Section 4.1 presents an in-depth discussion on the format, content and distribution of Project Reports, including the:

1. Preliminary Field Review Report,
2. Scope of Work Report,
3. Design Parameters Report, and
4. Plan-in-Hand Report.

These apply to all bridge rehabilitation projects.

22.6.3 Plan Preparation

Chapter Five discusses the preparation of plans for Department projects (e.g., content of individual sheets, scales, symbols). As applicable, bridge rehabilitation projects should be prepared consistent with these plan preparation conventions.

22.6.4 Roadway Design

The **Montana Road Design Manual** discusses the Department's roadway design criteria, and Sections 13.5 and 13.6 of the **Montana Structures Manual** discuss geometric design criteria specifically for highway bridges. The following discussion applies to geometric design considerations for bridge rehabilitation projects.

22.6.4.1 Horizontal/Vertical Alignment

Many existing bridges have alignments which do not meet MDT's current criteria for horizontal and vertical alignment. Except in rare cases, for bridge rehabilitation projects, it is unlikely to be cost effective to realign the bridge to correct any alignment deficiencies.

22.6.4.2 Roadway Cross Section

Section 13.5.4 presents information and criteria for the roadway cross section across a bridge, including:

- profile grade line,
- cross slopes and crowns,
- width,
- sidewalks,

- bikeways, and
- medians.

The application of these criteria to a bridge rehabilitation project will depend upon the Scope of Work and the practicality of meeting these provisions.

Specifically for roadway widths across bridges, MDT has produced the **Montana Bridge Design Standards**, which is a separate document. Officially adopted standards are the highest order control document over design criteria. The bridge designer should reference this document to determine MDT policies and criteria for bridge widths on existing bridges to remain in place (or bridge rehabilitation projects). Many documents within MDT commonly referred to as "standards" are in fact guides. Among these guides is the **Montana Road Design Manual**. Within some broad parameters, virtually all information in the **Montana Road Design Manual** is considered guidance. If descrepencies exist between the **Montana Bridge Design Standards** and the **Road Design Manual**, the **Montana Bridge Design Standards** is the controlling document. When deviating from the **Standards**, formal design exception approval from the Bridge Engineer must be secured.

22.6.5 Appurtenances

22.6.5.1 General

Section 15.5 discusses the various appurtenances that may be present on bridges:

1. bridge rails,
2. pedestrian rails,
3. bicycle rails,
4. fences,
5. utility attachments,
6. sign attachments, and
7. lighting/traffic signals.

With the exception of bridge rails (Section 22.6.5.2), the policies and criteria in Section 15.5 apply to bridge rehabilitation projects. For example, the bridge designer will refer to Section 13.5.4 "Fences" to determine if protective fencing across the bridge is warranted.

22.6.5.2 Bridge Rails

The need to revise existing bridge rails will be based upon the Department's "Montana Bridge Rail Policy and Practice," dated August 1988. This Policy is located at the end of Section 22.6. If the existing bridge rail will be replaced, the criteria in Section 15.5.1 for new bridge rails will apply.

For guardrail-to-bridge-rail transitions, the Road Design Section, in coordination with the bridge designer, is responsible for determining if any modifications are warranted.

22.6.6 Maintenance and Protection of Traffic During Construction

Work zone traffic control is an important element of a bridge rehabilitation project. The proposed strategy for maintaining traffic during construction could include alternating one-way traffic with signals, lane restrictions, median crossovers, or diverting the traffic to a detour route. See the **Montana Road Design Manual** for detailed information on Department policies and procedures.

Montana Bridge Rail
Policy and Practice

This policy and practice statement will provide additional clarification regarding the Montana Department of Transportation's bridge rail policy which the FHWA Division Office concurs in. Since the meeting of January 13, between MDT and the FHWA Division Office, Mr. Morgan has concurred in Mr. Loveall's additional guidelines to AASHTO's proposed guide specifications for bridge railings. The additional guidelines specifications state, in part, that railing designed and built subsequent to the institution of the 1964 Interim AASHTO Specifications railing provisions are not subject to replacement, provided its performance record is proven satisfactory. Existing Montana bridge rail that does not meet the 1964 specifications will be required to be upgraded to a crashworthy design.

Since the preface of the Guide Specifications clarifies that the Guide is for new bridges and for bridges being rehabilitated to the extent that railing replacement is obviously appropriate, therefore, on a case-by-case basis, bridge rails designed and constructed subsequent to the 1964 AASHTO's Specifications may be retained in service. Site-specific data and railing performance data will need to be reviewed to determine whether to replace or retrofit the existing bridge rail to a crashworthy configuration or leave the existing bridge rail in place.

If the bridge rail has been previously blocked out and only requires repairs, which are normally performed by MDT maintenance forces, then the rail can remain in place provided the rail will be repaired before the contract is let. The period between the field check stage and letting date should allow adequate time for the rail to be repaired.

Montana's Type No. 5 bridge rail with a 150-mm offset was designed prior to the 1964 interim specifications and, therefore, should be retrofitted. A blocked out thrie beam is currently being used. The Type No. 5 rail with a 75-mm offset to rail from curb was designed after the 1964 interim specifications. Therefore, this rail, contingent on a review of each site and performance data, can remain in place.

Montana's Type No. 3, Type No. 4, and concrete post bridge rails were designed prior to 1964. If these rails have not been blocked out, they should be replaced or retrofitted. The Type No. 3 rail, when within a Federal-aid project, is being replaced with a cast-in-place, concrete barrier. The T4 and concrete post bridge rails are retrofitted, with Division Office approval, by blocking out a thrie beam to the face of the curb.

An existing steel beam bridge rail, SBBR, that is blocked out to the curb can remain in place. Again, site-specific data regarding railing performance and condition of existing rail components will need to be evaluated to determine if the blocked out SBBR can be left in place. If the existing SBBR is not blocked out, then a cast-in-place concrete barrier rail will be constructed. The blocked out SBBR's first steel post will be modified to accept Montana's crashworthy approach rail. The blocked out SBBR will still be treated as a bridge rail to remain in place since this modification to the first bridge rail post is only for accepting a crashworthy approach rail.

An existing timber bridge rail that is blocked out to the curb satisfies the requirements of the 1964 provisions of AASHTO's Interim Bridge Railing Specifications. An existing timber bridge rail that is not blocked out to the curb does not satisfy these requirements. In accordance with previous correspondence from the FHWA Regional and Washington Offices, the precast concrete barrier, originally proposed by the MDT, is not an acceptable retrofit. A continuous, cast-in-place concrete barrier rail secured to the deck, proposed by the MDT, is an acceptable solution.

In accordance with accepted policy, bridge rails for new bridge construction or bridge replacement are being constructed to a crashworthy design. The FHWA Regional Office concurs with holding off of Montana's crash testing of the T5 rail with a 75-mm offset and the SBBR until the proposed guide specifications are adopted. The adopted specifications may not require crash testing of rails designed after the 1964 Interim Bridge Specifications.

The 1988 bid estimates for retrofitting bridge rails with a blocked-out thrie beam has averaged about $16 per lineal foot. Assuming traffic control to be 20% results in a total unit cost of $20 per lineal foot for blocking out with a thrie beam.

The 1988 bid estimates for placing a cast-in-place concrete barrier has averaged $85 per lineal foot. Assuming traffic control to be 20% results in a total unit cost of $102 per lineal foot for the concrete barrier.

For the remaining August through December 1988 lettings, the cost to upgrade bridge rail to a crashworthy configuration within Federal-aid projects is estimated to be $150,000.

<u>Summary of Guardrail Retrofit Policy by FHWA</u>:

1. <u>Montana SBR-T3</u>. Retrofit with cast-in-place concrete barriers.

2. <u>Montana SBR-T4</u>. Thrie-beam retrofit upon Division Office approval only. Dependent upon needed repairs, performance and other criteria.

3. <u>Montana SBR-T5</u>. Design prior to the AASHTO Interim 1964 (curb to rail spacing of 150-mm) — Use blocked-out thrie-beam retrofit.

   Design after the AASHTO Interim Specifications of 1964 (curb to rail spacing of 75 mm) — no retrofit needed. This system can remain as is.

4. <u>Montana SBBR</u>. If this rail system is blocked out to the face of the curb, no retrofit is needed. This will also depend upon the condition and the performance of the existing rail.

   If this rail system is not blocked out to the face of the curb, a cast-in-place concrete barrier rail retrofit is required. New approach rail corresponding to a crashworthy bridge rail is also required.

5. <u>Timber Rails</u>. If blocked out to the face of the curb, no retrofit is needed. This curb-rail system can be left in place, depending upon the condition and the performance.

   If the existing rail is not blocked out by a W-beam rail, then a cast-in-place concrete barrier rail retrofit is required. The approach rail also has to correspond to a crashworthy rail.

22.7 BRIDGE REHABILIATION LITERATURE (other than AASHTO documents)

22.7.1 FHWA Documents

1. Extending the Service Life of Existing Bridges by Increasing Their Load-Carrying Capacity, 1978.
2. Seismic Retrofitting Guidelines for Highway Bridges, 1983.
3. Inspection of Fracture-Critical Bridge Members; supplement to the Bridge Inspector's Training Manual, 1986.
4. Seismic Design and Retrofit Manual for Highway Bridges, 1987.
5. Seismic Design Retrofit Manual for Highway Bridges, 1989.
6. Economical and Fatigue-Resistant Steel Bridge Details, 1990.
7. Bridge Inspector's Training Manual, 1991.
8. Evaluating Scour at Bridges, 1991

22.7.2 TRB Documents

1. National Cooperative Highway Research Program (NCHRP) Report 141: *Bridge Deck Joints*, 1989.
2. NCHRP Report 206: *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, 1979.
3. NCHRP Report 243: *Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads*, 1981.
4. NCHRP Report 248: *Elastomeric Bearings Design, Construction and Materials*, 1982.
5. NCHRP Report 271: *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*, 1984.
6. NCHRP Report 280: *Guidelines for Evaluation and Repair of Damaged Prestressed Concrete Members*, 1985.
7. NCHRP Report 293: *Methods of Strengthening Existing Highway Bridges*, 1987.
8. NCHRP Report 298: *Performance of Elastomeric Bearings*, 1987.
9. NCHRP Report 299: *Evaluation Procedures for Steel Bridges*, 1987.
10. NCHRP Report 301: *Load Capacity Evaluation of Existing Bridges*, 1987.
11. NCHRP Report 302: *Fatigue and Fracture Evaluation for Rating Riveted Bridges*, 1987.
12. NCHRP Report 333: *Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges*, 1990.
13. NCHRP Report 336: *Distortion-Induced Fatigue Cracking in Steel Bridges*, 1990.
14. NCHRP Synthesis of Highway Practice 41: *Bridge Bearings*, 1977.
15. NCHRP Synthesis of Highway Practice 88: *Underwater Inspection and Repair of Bridge Substructures*, 1981.
16. NCHRP Synthesis of Highway Practice 136: *Protective Coatings for Bridge Steel*, 1987.
17. NCHRP Synthesis of Highway Practice 141: *Bridge Deck Joints*, 1989.

22.7.3 Other Documents

1. American Concrete Institute (ACI), *Guide to Joint Sealants for Concrete Structures*, 1977.

2. ACI, *Guide for Repair of Concrete Bridge Superstructures*, 1980.

3. Wasserman, Edward P., "Jointless Bridge Decks," *Engineering Journal*, American Institute of Steel Construction (AISC), Third Quarter, 1987.

Table of Contents

Chapter Twenty-three

MISCELLANEOUS STRUCTURES

The primary responsibility of the Bridge Bureau is to design highway bridges. However, the Bureau is also responsible (partially or entirely) for the structural design of a variety of other highway-related structures. Chapter Twenty-three briefly discusses these.

23.1 WALLS

23.1.1 Responsibilities

The Geotechnical Section, Road Design Section and Bridge Bureau typically collaborate on the design of walls. The Bridge Bureau is generally responsible for the structural design of cast-in-place, concrete retaining walls. The Road Design Section determines the height of retaining wall.

The Geotechnical Section is generally responsible for the design of reinforced earth structures, bin walls and gabions.

23.1.2 Conventional Retaining Walls

Reference: LRFD Article 11.6

Retaining walls are essentially the same as abutments with the exception of the absence of bridge-bearing loads and the handling of live-load surcharge (LS) as specified in LRFD Article 3.11.6.2. As such, loads on retaining walls should be determined as specified in Section 19.1.2, and the appropriate design and detailing provisions of Section 19.1.3 should be applied.

Foundations supporting retaining walls should be selected and designed according to Chapter 20. Conventional retaining walls are generally of the reinforced concrete type. The resistance of reinforced concrete retaining walls in terms of flexure and shear should be determined according to Chapter 16.

23.1.3 Prefabricated Earth Retaining Systems

Reference: LRFD Articles 11.8, 11.9 and 11.10

Prefabricated earth retaining systems include, among others, mechanically stabilized earth (MSE) walls, prefabricated modular walls, anchored walls, etc., all defined in LRFD Article 11.2. Typically, the wall contractor is responsible for the design of the wall system, checked by the Geotechnical Section, which typically selects the wall type.

MSE walls should be considered as alternatives to conventional retaining walls where substantial total or differential settlements are anticipated. The decision to use a conventional retaining wall or a MSE wall (or other prefabricated earth retaining system) will be made by the Geotechnical Section.

23.2 OTHER STRUCTURE TYPES

23.2.1 Structural Supports for Signs, Luminaires and Traffic Signals

The Department has adopted the use of the AASHTO **Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals**. The MDT Traffic Engineering Section is primarily responsible for these supports, and the Department has developed standard designs which will apply in most cases.

In certain cases, the Bridge Bureau will become involved in the design of structural supports for these roadside appurtenances. See the following for a description of the Bridge Bureau's involvement:

1. Coordination. Section 3.1.6.1 discusses supports for sign structures, traffic signals and luminaries, and it discusses the coordination between the Bridge Bureau and Traffic Engineering Section.

2. Cantilever/Overhead Signs. Section 18.9.1 of the **MDT Traffic Engineering Manual** specifically discusses the structural design of cantilever and overhead sign structures, including the role of the Bridge Bureau.

23.2.2 Buried Structures

Buried structures include reinforced concrete culverts, metal pipes, structural plate pipes, etc. The structural design of buried structures is based on Section 12 of the LRFD Specifications. The MDT Hydraulics Section is primarily responsible for these drainage appurtenances, and the Department has developed standard designs which will apply in most cases. Occasionally, the Hydraulics Section may request the Bridge Bureau to check the structural adequacy of a proposed or existing culvert.

23.2.3 Sound Barriers

The structural design of sound barriers is based on the AASHTO **Guide Specifications for Structural Design of Sound Barriers (1989 Edition)**.

23.2.4 Miscellaneous

Occasionally, the Bridge Bureau is requested to provide other structural engineering services for the Department. The engineer responsible for this work must determine the appropriate design code for the assignment.

Table of Contents

Chapter Twenty-Four
CONSTRUCTION OPERATIONS

24.1 GENERAL

The Bridge Design Section of the Bridge Bureau continues to be involved during construction to ensure the designer's intent is followed. Frequently, issues arise because of construction errors or materials problems that could compromise the performance or longevity of the finished product. Additionally, because the contractor often wishes to place construction equipment on bridges that MDT wishes to remain in service, the Section reviews engineering submittals from the contractor for the use of large equipment on bridges. Design intent is ensured by approval of materials, fabrication, etc., of structural components of bridges.

The Bridge Design Section is also contacted for advice on out-of-specification materials and construction errors. Because the Section is frequently expected to respond very quickly to construction problems, these problems frequently become the highest priority.

24.2 SHOP DRAWINGS

The contractor must submit drawings that show how, and of what materials, structural components will be fabricated. These drawings are widely used throughout the Department, and explicit procedures for submittal, review and approval have been developed.

24.2.1 General

The Bridge Bureau has the responsibility for reviewing and approving fabricator's shop drawings for structural steel, prestressed beams and structural timber.

24.2.2 Responsibility

A position in the Bridge Management Section is used full time to provide this function.

24.2.3 Procedures

The Shop Drawing Reviewer has written procedures to ensure accountability, consistency and a timely review of submitted material.

24.2.3.1 Shop Drawing and Material Submittal

The requirements for shop drawings submittals are contained in the **Montana Standard Specifications for Road and Bridge Construction**.

24.2.3.2 Copy Distribution

The distribution of approved shop drawings is as stated in the written shop drawings procedures.

24.2.3.3 Approval Level

The individual tasked with reviewing shop drawings is delegated approval authority for the shop drawings.

24.2.3.4 Bridge Bureau Review

The Shop Drawings for prestressed girders are submitted with calculations from the fabricator to validate that the proposed girder strand configuration and concrete strength is adequate for the purposes of the project. The Shop Drawing Reviewer gives these calculations to the appropriate Bridge Area Engineer for review and approval. Calculations currently take the form of computer output from one of several approved prestressed beam design programs. Currently, these programs are Con-Span and BT Beam. See Chapter Twenty-five. The designer must review data input for the computer runs and verify that the output indicates correct beam stresses and adequate ultimate moment capacity. The Shop Drawing Reviewer has approval authority for the drawings, and no additional review of shop drawings occurs within the Bridge Bureau.

24.2.3.5 Checklists

The individual that reviews shop drawings has developed a series of shop drawing approval checklists that are specific to the material and type of component being fabricated.

24.3 FIELD ISSUES

Bridge Bureau structural engineers are on-call to Construction Bureau and District construction staff for consultation on materials that do not comply with specification materials, variations in construction tolerances that are beyond normal and accepted practice, and construction mistakes. If problems arise during construction, Bridge Design Section staff must frequently travel to the project site to observe, gather information and discuss options with District construction staff. Bridge Bureau personnel work through the District construction staff and do not interface directly with the contractor's staff.

24.3.1 Construction Realities

From a practical standpoint, it is not realistic to expect that the contractor can construct the project precisely to the dimensions set forth in the construction plans. Reasons include:

1. The allowable deviation from dimensions in the plans should be determined based on the impact of the deviation on the structural integrity or the appearance of the structure. For example, a deviation of 10 mm might be of no consequence in a foundation, but this deviation might seriously weaken a thin slab.

2. Deviations from the specified dimensions may be greater on a structural component not exposed to view.

3. Preferably, any deviation should be too thick rather than too thin.

A contractor builds from the bottom up. Not so obvious is the fact that, because of flowing water or other difficult site conditions, when piers or bents are initially laid out by the surveyors, they might not be very precise. The construction process that gives the least risk to the contractor is to wait until one bent or pier is constructed to establish a firm centerline bearing at centerline bridge, and then build the rest of the bridge to fit. In this way, one is assured that the superstructure will fit the substructure. Tolerances will become progressively tighter as the bridge is constructed and, finally, tolerances of 3 mm on deck flatness in any 3000-mm radius are achieved.

The District construction field personnel will have the final decision-making authority on whether or not the contractor is constructing an acceptable product If requested, the Bridge Bureau will provide input and advice on a case-by-case basis.

24.3.2 Technical Assistance

The Bridge Design Section provides assistance as requested for the construction of structural items.

24.3.2.1 Effects of Construction Activity

By its very nature, much construction machinery tends to be compact and very robustly constructed. Often, a piece of construction machinery has a weight that seems out of proportion to its size. A piece of equipment may be tasked to hoist loads, which adds to the effective weight of a piece of machinery. The contractor may wish to place heavy construction machinery on a partially completed structure, or may wish to place heavy construction machinery on a bridge that MDT will own and use at the conclusion of the work.

Under these circumstances, the contractor must retain an outside engineering firm to conduct an appropriate structural analysis and submit a summary report of the construction machinery's effect on the bridge. These engineering calculations obviously need to use an appropriate machine weight, such as data presented in the manufacturer's literature or, better yet, certified scale tickets. The report that the engineering firm prepares must specifically define an operational "envelope" of loads and conditions within which the machinery must operate.

24.3.2.2 Contractor Submittals to Correct Fabrication/Construction Errors

Not infrequently, the Bridge Design Section is contacted to review the technical merits of a technical submittal from a contractor attempting to salvage an undesirable situation or attempting to salvage whatever is salvageable out of a misfabricated component or perhaps a bent or pier that was constructed in the wrong location.

Determine relevant facts and develop solutions to problems encountered during construction when it appears to the District construction staff that the designer's intent is physically unconstructable.

24.3.2.3 Change Orders

The Bridge Design Section receives and comments on all bridge-related construction change orders and works with the Construction Bureau and the District construction staff to resolve issues.

24.3.2.4 Value Engineering

Evaluate Value Engineering proposals for structural items. Although Bridge Plans represent one solution to the structural problem, the solution presented may not be the most economical for a particular contractor because each contractor has unique equipment and staff expertise. A contractor may wish to submit a proposal to value engineer alternative methods or materials. The Bridge Design Section will be asked to review any value engineering submittal that concerns bridge issues. Any proposed product must be at least equal in functionality and longevity as that shown in the contract documents.

24.3.2.5 Evaluate Product Submittals

The designer may on occasion specify a type of product that the District construction staff is unfamiliar with. The Bridge Design Section may be requested to evaluate a product to determine if the product meets the designer's intent and the contract documents.

24.4 STANDARD AND SUPPLEMENTAL SPECIFICATIONS

The Bridge Design Section works closely with the Specifications Section of the Construction Bureau for the development and modification of the **Montana Standard Specifications for Road and Bridge Construction**. The Bridge Design Section's core areas of interest are in Division 550 – Structures and Division 700 – Materials.

Table of Contents

Table of Contents
(Continued)

Section **Page**

Chapter Twenty-Five
COMPUTER PROGRAMS

25.1 GENERAL

25.1.1 Introduction

The Bridge Bureau uses many computer programs for structural design, which can provide significant benefits. These include the capability of quickly analyzing several alternative designs (i.e., simulation capabilities), of reducing the probability of mathematical errors and for saving time by avoiding laborious hand calculations. However, the user of any computer program must consider the following:

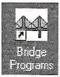
1. Judgment and experience are critical to the proper interpretation of the computer outputs.
2. The user should, after the computer run, recheck inputs for accuracy.
3. The user should carefully check all output to ensure that answers are reasonable and logical and that there are no obvious errors. The check should include an equilibrium check in structural applications, for example, verifying that the sum of the applied loads equals the sum of the reactions.
4. The user should be familiar with the advantages and limitations of each program.

25.1.2 Chapter Content

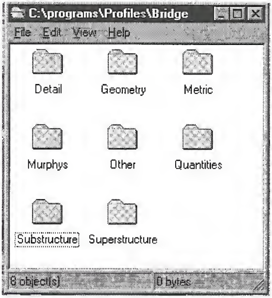
Chapter Twenty-five presents the majority of the computer programs used by the Bridge Bureau. For most programs, the Chapter provides a brief description of the program's applications, its inputs and its outputs. The Chapter is segregated into those programs that have been developed internally by the Department and those programs developed external to the Department.

25.1.3 Access to MDT Programs

The MDT Library of Computer Programs is accessed through the following icon on most of the Bureau's Computers:



If you do not have this icon, the user can access the Library by clicking on "Start" then "Programs." Look for the Bridge Program listing in the column of options. By clicking on that icon, the user can call up the following selection of available folders:



Chapter Twenty-five describes most of the commonly used programs in these folders. For information on programs not described in Chapter Twenty-five, contact the Bridge Area Engineer.

25.2 MDT PROGRAMS

See Figure 25.2A.

Name	Location
1. Bridge End Stations	Metric Version - Bridge Programs\Metric\Bridge End Stations English Version - Bridge Programs\Geometry\ Bridge End Stations
2. Wingwall Lengths on Bents with Skew	Bridge Programs\Geometry\Wingwall Lengths on Bents with Skew (*English Units*) Bridge Programs\Metric\Wingwall Lengths on Bents with Skew
3. COGO87	Bridge Programs\Geometry
4. 10th Points Program	Bridge Programs\Geometry
5. D Depths for Spans on a Vert. Curve	Metric – Bridge Programs\Metric\D Depths for spans on a Vert. Curve English – Bridge Programs\Geometry\D Depths for spans on a Vert. Curve
6. Vertical Curve Elevations on CL of Tangent Roadways with Grade Constant	Bridge Programs\Metric\Vertical Curve Elevations
7. Bridge Beam Seat and Profile Grade Elevations	Bridge Programs\Geometry
8. Prestress Beam Design — Metric AASHTO 16th	Bridge Programs\Metric\Prestressed Beam Design
9. Prestress Beam Design — Metric LRFD	Bridge Programs\Metric\LRFD Prestressed Beam Design
10. Prestressed Bulb-'T' Beam Design	Bridge Programs\Superstructure\Prestressed Bulb-Tee Beam Design
11. Prestressed Tri-deck Beam Design	Bridge Programs\Superstructure\Tri Deck
12. Slab Design	Bridge Programs\Superstructure\Slab Design
13. Distribution of Flexural Steel	Metric – Bridge Programs\Metric\Distribution of Flexural Steel English – Bridge Programs\Substructure\Distribution of Flexural Steel
14. Reactions at End and Intermediate Bents	Bridge Programs\Metric\Reactions at End and Intermediate Bents
15. Round Column Design Using Strength Methods	
16. Rectangular Column, Biaxial Bending — Strength Design	Bridge Programs\Substructure
17. Soil Spring and Footing Stiffness	Bridge Programs\Other\footing.exe
18. Slab Bar Lengths on a Curved Bridge	Bridge Programs\Metric\Curved Slab Bar Lengths
19. Skewed Slab Bar Lengths	Bridge Programs\Metric\Bar Lengths (Skewed Slab)
20. Reinforcing Steel Weight	Bridge Programs\Metric\Reinforcing Steel Weight
21. Prestressed Beam Diaphragm Volume	Bridge Programs\Metric\Diaphq2m
22. End Bent Quantities	Bridge Programs\Quantities\End Bent Quantities for English or Bridge Programs\Quantities\Metric\End Bent Quantities for Metric
23. Area Centroid Moment of Inertia	Bridge Programs\Geometry

MDT COMPUTER PROGRAMS

Figure 25.2A

Title: Bridge End Stations

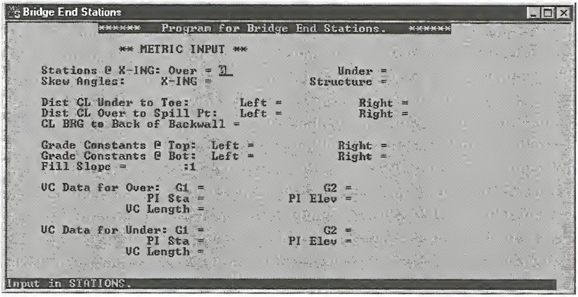
Units: Available in both Metric and English Units

Location: Metric Version - Bridge Programs\Metric\Bridge End Stations
English Version - Bridge Programs\Geometry\Bridge End Stations

Description:

Program calculates the required structure length based on the feature crossed, fill slopes, bridge geometry and roadway vertical curve information. The program rounds the bridge length to the nearest meter or foot and calculates the Station and Profile Grade Elevation at the bridge ends and center. This program is very helpful in preliminary design to help determine a bridge type and size. If used for preliminary design, the input values below can be approximated.

Input Screen:



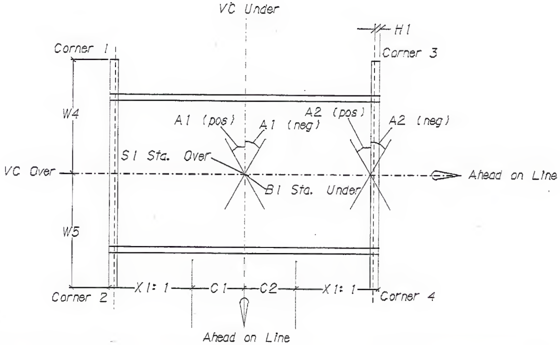
Input Definitions:

Terms:
- Over: Bridge that is being sized.
- Under: This could be a highway or interstate road, railroad tracks or stream crossing that the bridge (Over) is spanning.

Required information: A wing length based on the estimated beam type must be determined to calculate some of the input values below. The Wingwall Length program can be used.

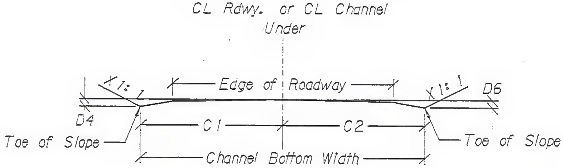
S1 = ℄ Station @ Crossing Over = _____ Stations (ex: 2+50.50 = 2.5050 Stations)

B1 = ℄ Station @ Crossing Under = _____ Stations



Skew angle positive if right side is ahead on line, negative if left side is ahead on line.

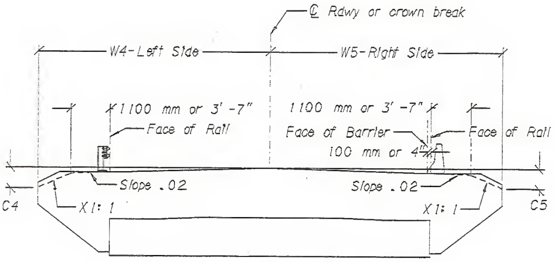
A1 = Skew angle of Crossing Under = _____ degrees
A2 = Skew angle of Structure Over = _____ degrees



C1 and C2 are measured normal to ℄ Under.

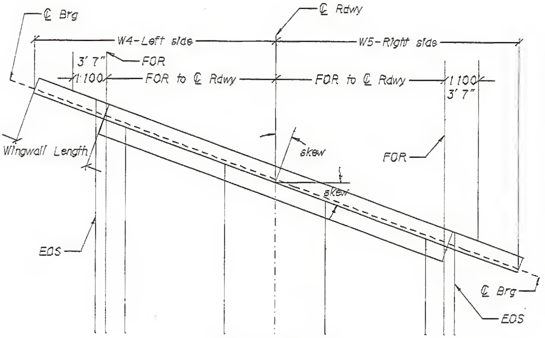
Roadway/Railroad: Measure from the Toe of Slope to ℄ for each side
Stream crossing: C1 and C2 are typically half the channel bottom width (Hyd Report)

C1 = Left distance from ℄ under to toe = _____ m or ft (Always positive)
C2 = Right distance from ℄ under to toe = _____ m or ft (Always positive)



All dimensions normal to CL Rdwy Over
(Looking Ahead on Line)

For Skewed Bridges:



Looking Ahead on Line

W4 = Left perpendicular distance from ℄ over to spill point = _____ m or ft (Usually neg.)
W5 = Right perpendicular distance from ℄ over to spill point = _____ m or ft (Usually pos.)

H1 = ℄ Brg. to fill face of backwall = _____ m or ft (Usually 0.4 m or 1.25 ft)
(Flat Slab = 180 mm or 0.5833 ft)

(To use program with turned back wings, enter H1 as the distance from ℄ Brg to back of the turned back wingwall. The dimension W4 and W5 will also change measured from ℄ Rdwy to the outside edge of wingwall.)

C4 and C5 below are measured at the edge of wingwall on the fill face of Backwall
C4 = Left grade constant @ top = _____ m or ft *
C5 = Right grade constant @ top = _____ m or ft *

* Sign convention: Negative if elevation of spill point is below Profile Grade @ ℄ Roadway
Positive if elevation of spill point is above Profile Grade @℄ Roadway

**D4 = Left grade constant @ bottom = _____ m or ft
**D6 = Left grade constant @ bottom = _____ m or ft

** Sign convention: Negative if elevation of toe is below Profile Grade @ ℄ Roadway under.
Positive if elevation of toe is above Profile Grade @ ℄ Roadway under.

X1 = Fill Slope = _____ (Usually 2, for 2:1 slope)

Vertical Curve — Over

G1 = Grade between PC & PI	=	_____ %	
G2 = Grade between PI & PT	=	_____ %	
PI = Station @ PI	=	_____ Station	
E1 = Elevation @ PI	=	_____ m or ft	
L1 = Length of Vertical Curve	=	_____ Stations	(Ex: 300 m = 3.00)

Vertical Curve — Under *

G1 = Grade between PC & PI	=	_____ %
G2 = Grade between PI & PT	=	_____ %
PI = Station @ PI	=	_____ Station
E1 = Elevation @ PI	=	_____ m or ft
L1 = Length of Vertical Curve	=	_____ Stations

* For a stream crossing, the above information can be taken from the hydraulic report. G1 and G2 — the Channel Slope converted to %. P1 — usually same as B1 above. E1 — the Channel Bottom Elevation. L1 — anything wider than the bridge abutments.

Verification of Results:

Verify that the correct input values were used and the length of the bridge seems reasonable.

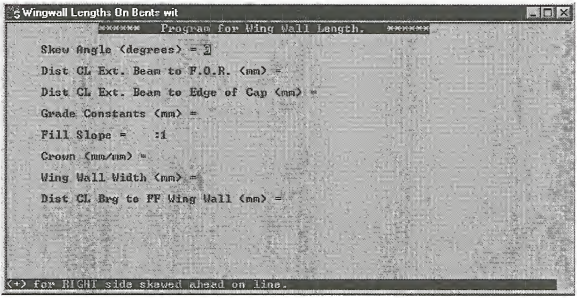
Title: Wingwall Lengths on Bents with Skew

Location: Bridge Programs\Geometry\Wingwall Lengths on Bents with Skew (*English units*)
Bridge Programs\Metric\Wingwall Lengths on Bents with Skew

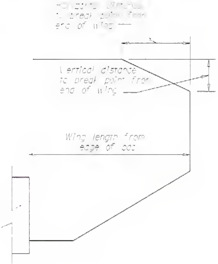
Description: Calculates the wingwall length for a stub abutment.

Input:

- Skew angle (left ahead positive, right ahead negative).
- Perpendicular distance from centerline of exterior beam to face of rail (FOR).
- Distance along the skew from centerline of exterior beam to the end of the cap.
- Grade constant (distance from top of slab at centerline exterior beam to top of cap).
- Fill slope (?:1).
- Crown or superelevation (m/m slope upward from exterior beam to centerline roadway is positive, downward slope is negative).
- Wingwall "width" (actually thickness).
- Distance from centerline of bearing to the fill face of the backwall.



Output:



Title: COGO87

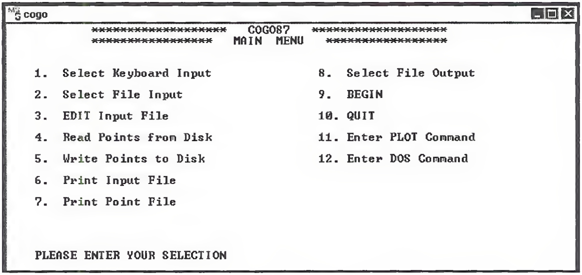
Location: Bridge Programs\Geometry

Description:

Program calculates coordinates and stations for geometric layouts. Refer to the published Manual for further information.

An alternative program is GEOPAK COORDINATE GEOMETRY.

Input:



Verification of Results:

Verify that the correct input values were used. Check for reasonable results.

Title: 10th Points Program

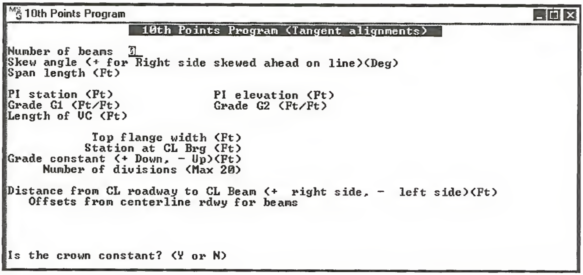
Location: Bridge Programs\Geometry

Description:

Program calculates elevations at 10th points for all girders within a span. Inputting the top flange width and haunch depth at the center of the beams allows elevations to be calculated at both sides of the top flange. This information is used during construction to set deck forms. Elevations can be calculated for a normal crown or superelevation transition. The program is limited to alignments on a horizontal tangent.

The program was written to work in feet but will work with meters because the units used in all calculations have consistent units. For the correct stations to be calculated, lengths should be adjusted for State Plane Coordinates.

Input:



Verification of Results:

Verify that the correct input values were used. Hand check an elevation output for verification of results.

Title: D Depths for Spans on a Vert. Curve

Units: Available in both Metric and English units.

Location: Metric – Bridge Programs\Metric\D Depths for spans on a Vert. Curve
English – Bridge Programs\Geometry\D Depths for spans on a Vert. Curve

Description:

The output from this program is used to calculate the correction to the camber based on the vertical curve for concrete or steel girders. Output is determined as follows:

The program calculates elevations at 10th points along the string line within a specified span length. The string line is the straight line between the beginning and ending station entered into the program. At each 10th point, the difference between the Profile Grade Elevation and String Line Elevation is calculated.

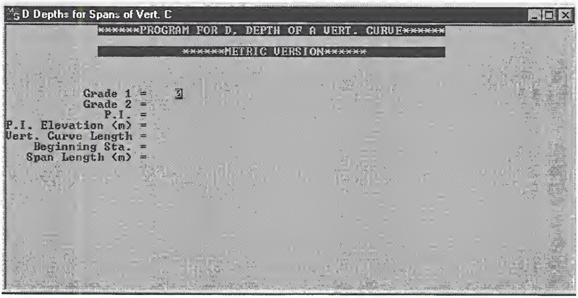
The Difference (mm) is used to determine varying haunch depth for concrete and steel girders as well as field splice locations for steel girders.

When using this program for steel girder design, the user should provide a Table of Camber Information (see Structures Manual p. 5.4(26)). This table indicates the amount of camber, to the nearest millimeter, that will be necessary for each girder at the 10th points of each span and at the field splices.

A Girder Camber Diagram is also necessary to illustrate the location of the deflections for each tenth point on the span, the location of field splices, the string line slope and the vertical curve offset.

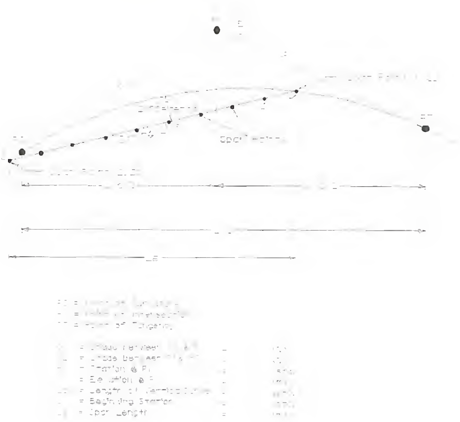
Finally, a Longitudinal Section diagram should be included in design. The Longitudinal Section illustrates an elevation view of the girder and bearing placement at each bent. It is important that the diagram include the offset dimension and the string line slope.

Input:



Output:

Graphical Description of Input and Output:



To calculate the String Line Slope, use the following equation:

$$\text{String Line Slope} = \left(\frac{[\text{Profile Grade Elev.@(Span Pt. 1.0)}] - [\text{Profile Grade Elev.@ (Span Pt. 0.0)}]}{\text{Ls}} \right) * 100$$

Verification of Results:

Verify that the correct input values were used. Hand checks can be done for verification of results.

Title: Vertical Curve Elevations on CL of Tangent Roadways with Grade Constant

Location: Bridge Programs\Metric\Vertical Curve Elevations

Description:

The program calculates vertical curve elevations at specific stations on a tangent alignment.

- Input the grade entering the curve (Grade 1) and the grade exiting (Grade 2) in percent ("+" for rising grade left to right and "-" for declining grade left to right).

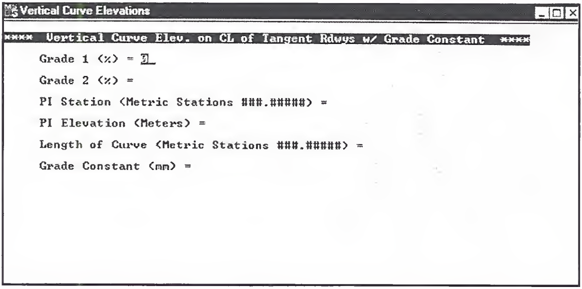
- Input the vertical curve's PI Station and Length in stations.

- Input the Grade Constant if needed. The grade constant will provide elevations at a constant distance above or below the gradeline input above.

Once the data is input, elevations can be displayed for specific stations for a series of elevations at constant increments and for a series of elevations at varying increments.

Choosing the "*Quit Application*" option from the menu displays selected elevations in a printable format.

Input:



Verification of Results:

Verify that the correct input values were used. Code the program to output a point of known elevation or hand check a minimum of one elevation as output for verification of results.

Title: Bridge Beam Seat & Profile Grade Elevations

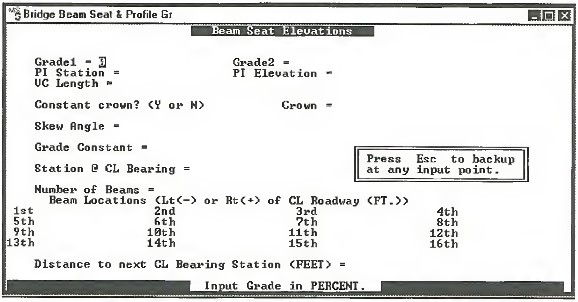
Location: Bridge Programs\Geometry

Description:

Program calculates beam seat elevations for all girders within a span. Multiple spans can be input. Elevations can be calculated for a normal crown or superelevation transition. The program is limited to alignments on tangent in the horizontal plane.

The program was written to work in feet but will work with meters because the units used in all calculations have consistent units. For the correct stations to be calculated, lengths should be adjusted for State Plane Coordinates.

Input:



Note: Input guidance is provided on the bottom line of the menu.

Verification of Results:

Verify that the correct input values were used. Hand check an elevation output for verification of results.

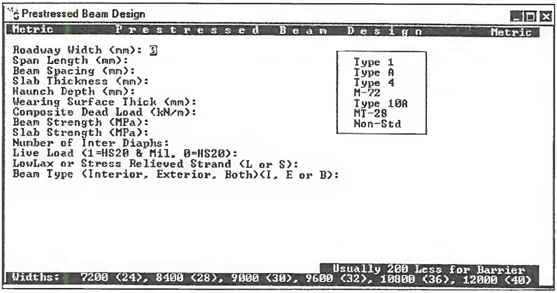
Title: Prestress Beam Design — Metric AASHTO 16th

Location: Bridge Programs\Metric\Prestressed Beam Design

Description:

The program designs Prestressed I-Girder and Non-standard sections in accordance with the AASHTO 1996 Standard Specifications, 16th Edition with the 1999 Interims. The program allows the designer to quickly determine the optimum girder spacing for a given span length, and it provides the typical design parameters needed for the erection plan. The program assumes and provides a strand pattern based on a 50-mm grid up to a maximum number for each girder consistent with the capacity of local suppliers. The program also is set to use concrete strengths consistent with the typical MDT allowable. The program will toggle between composite dead loads for both the T101 bridge rail and concrete barrier rail, and it includes the weight of a future wearing surface.

Input:



Note: *Input Road Width, Span Length and assumed Beam spacing in mm. Input slab strength in MPa using regional parameters (28 or 31). The remaining input values will automatically default unless overridden.*

Verification of Results:

- Verify that the correct input values were used.
- Verify that the transfer strength will allow daily bed rollover.
- Verify that a possible strand pattern was found and is reasonable.
- Note the controlling design (Interior or Exterior) and denote the controlling design moment and stresses for the plans.
- If the trial design fails, revise the design and rerun another trial.

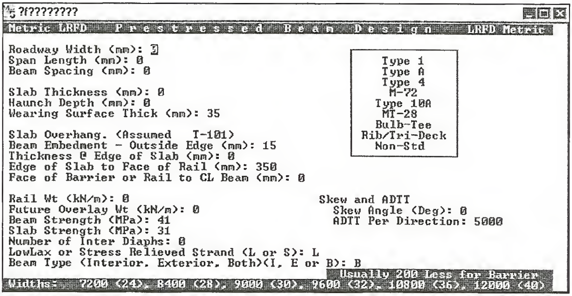
Title: Prestress Beam Design — Metric LRFD

Location: Bridge Programs\Metric\LRFD Prestressed Beam Design

Description:

The program designs Prestressed I-Girder, Bulb-Tee, Tri-Deck and Non-standard sections in accordance with the AASHTO 1998 LRFD Specifications. The program allows the designer to quickly determine the optimum girder spacing for a given span length, and it provides the typical design parameters needed for the erection plan. The program assumes and provides a strand pattern based on a 50-mm grid up to a maximum number for each girder consistent with the capacity of local suppliers. The program also is set to use concrete strengths consistent with the typical MDT allowable. The program will toggle between composite dead loads for both the T101 bridge rail and concrete barrier rail, and it includes the weight of a future wearing surface.

Input:



Note: Input Road Width, Span Length and assumed Beam spacing in mm. Input slab strength in MPa using regional parameters (28 or 31). The remaining input values will automatically default unless overridden.

Verification of Results:

- Verify that the correct input values were used.
- Verify that the transfer strength will allow daily bed rollover.
- Verify that a possible strand pattern was found and is reasonable.
- Note the controlling design (Interior or Exterior) and denote the controlling design moment and stresses for the plans.
- If the trial design fails, revise the design and rerun another trial.

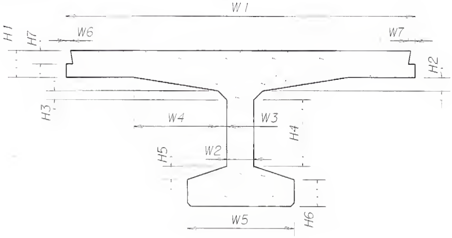
Title: Prestressed Bulb-'T' Beam Design

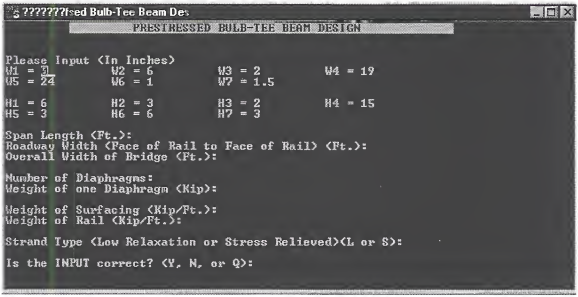
Location: Bridge Programs\Superstructure\Prestressed Bulb-Tee Beam Design

Description:

Given section dimensions and span requirements (see screen view below), the program designs the beam section necessary to meet them.

Input: Define the beam section by referring to the diagram.





Output:

The program provides section properties for the section (moment of inertia, section modulus, centroid location), dead and live load moments, dead load deflections, and allowable and actual stresses at transfer and in final configuration. The program generates stresses and deflections each for an interior beam and for an exterior one. It also generates a possible strand pattern for each one.

Verification:

Verify data input.

Title: Prestressed Tri-deck Beam Design

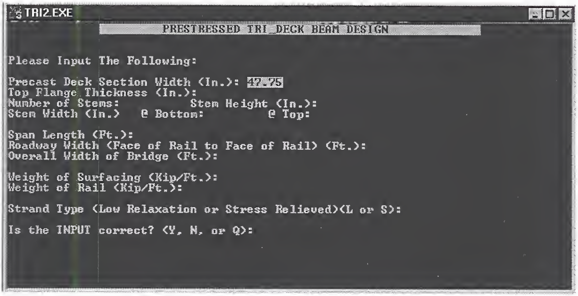
Location: Bridge Programs\Superstructure\Tri Deck

Description:

Given section dimensions and bridge properties, the program designs a beam section to meet the requirements.

Input:

Section dimensions, bridge dimensions, and future wearing surface and rail weight.



Output:

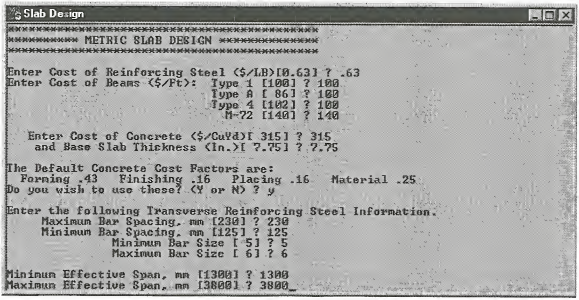
The program provides section properties for the section (moment of inertia, section modulus, centroid location), dead and live load moments, dead load deflections, and allowable and actual stresses at transfer and in final configuration. It also generates a possible strand pattern.

Title: Slab Design

Location: Bridge Programs\Superstructure\Slab Design

Input:

Enter the appropriate costs. Note that the default costs are not current.



Output:

The program generates a table of possible deck thicknesses at different effective slab spans and reinforcement patterns for each. It generates a similar list for each type of prestressed beam listed in the screen view. It offers an option to filter the list to show only the most economical sections. Note that the unfiltered list runs twenty-six pages.

Title: Distribution of Flexural Steel

Units: Available in both Metric and English Units

Location: Metric Version - Bridge Programs\Metric\Distribution of Flexural Steel
English Version - Bridge Programs\Substructure\Distribution of Flexural Steel

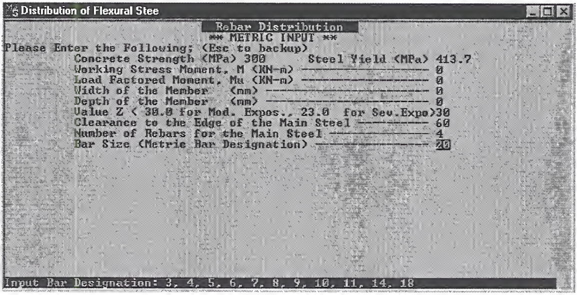
Description:

The program calculates the primary steel required for a rectangular concrete flexural member. Assume the initial flexural steel and check against the design moments for the section. Program calculates the cracking moment, area of steel required by load factored moment and tensile stresses in mild steel. Iteration determines the final area of flexural steel.

The structural analysis is based on the requirements of the AASHTO **Standard Specifications for Highway Bridges, Sixteenth Edition** with `97 Interims (Section 8.16.8.4).

The structural analysis also applies to the requirements of the AASHTO **LRFD Bridge Design Specifications**, Second Edition 1998 with `01 Interims (Section 5.7.3.2).

Input Screen:



Input Definitions:

Required information: Design moments for the analyzed section. The working stress moment equals the Service Limit State moment.

Concrete Strength, **F`$_c$** = ______ MPa or ksi (Default 21 MPa or 3 ksi)
Steel Yield, F$_y$ = ______ MPa or ksi (Default 413.7 MPa or 60 ksi)

Working Stress Moment, **M =** ______ kN-m or ft-kips
Load Factored Moment, **Mu =** ______ kN-m or ft-kips
Width of Member, **W =** ______ mm or inches
Depth of Member, **D2 =** ______ mm or inches
Crack Width Parameter, **Z =** ______ 30 kN/mm or 170 kips/in for moderate exposure (Default)
23 kN/mm or 130 kips/in for extreme exposure
Clearance to the Edge of the Main Steel, **D3 =** ______ mm or inches (Default 60 mm or 2.5 in)
Number of Rebars for the Main Steel, **N =** ______ bar fractions acceptable (Default 4 bars)
Enter the Bar Size, **S =** ______ (Default #6) (Allowed bar designations are English only)

Output Screen:

Error messages that require parameter modifications and rerun of the program due to failure of code requirements:

!!! The Area of Steel is Insufficient Check Section 8.17.1.1-2 AASHTO
!!! The Area of Steel is Insufficient for Mu.
!!! The actual working stress is too high.

```
Distribution of Flexural Stee
          Load Factored Moment (Mu) ------   300.00 KN-m
          Width of the Member is -------      1000.00 mm.
          Depth of the Member is -------     1000.00 mm.
          Z (30.00 for Mod Exp 23.00 for Sev Exp) is  30.0 MN/m
          Clearance to Main Steel =   63 mm.
          The Main Steel is -----      4.000 # 7 BARS

R E S U L T S
"As" provided is ----------     1548.384 mm.Sq.   (  4.000 # 7 BARS)
"As" required for 1.2*Mcr --    1692.123 mm.Sq.   (  4.371 # 7 BARS)
"As" required by Mu --------    879.910 mm.Sq.   (  2.273 # 7 BARS
                                                 * 4/3 =    3.031 # 7 BARS)
Phi * Mn -----     523.446 KN-m
Actual Mu ----     300.000 KN-m      P (RHO)    -------------   0.001672
1.2 * Mcr ----     570.989 KN-m     .75Pb (RHO Balanced) ---   0.016279


Allowable Working Stress is -------    243.84 MPa
Actual Working Stress is ----------    184.15 MPa

        End of Output.

          Press any key to continue.
```

Verification of Results:

Check the input values. Perform separate hand calculations to determine the required area of steel.

Title: Reactions at End and Intermediate Bents

Location: Bridge Programs\Metric\Reactions at End and Intermediate Bents

Description:

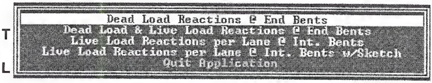
The program will calculate Dead Load reactions at the girder ends of a simple-span, prestressed girder bridge. It also has a routine that calculates LL reactions per lane at end or intermediate bents. The Program has four options: 1) DL Reactions at End Bents, 2) Combined DL and LL reactions per lane at end bents, 3) LL reactions per lane at intermediate bents, and 4) LL reactions per lane at intermediate bents with sketches.

Limitations:

- HS-20 loading, AASHTO Standard Specifications, 16th Edition.
- For simple-span designs only.
- The DL routine does not handle steel beams.

Input:

Need basic dimensions of spans, roadway width and backwall for dead load reactions. Input is self-explanatory from the menu. Program provides a toggle for concrete barrier rail or T101 rail.



Verification of Results:

Verify that the correct input values were used.

Note: Dead Load Reactions output is kN/Girder for interior and exterior girders at the top of the cap. Cap weight is not included in the output.

Live Load Reactions output is kN/Lane at the end reaction. Sketches show if truck or lane loading controls and placement of the loads. End reactions need to split out to the individual girders as specified in Section 3 of the Standard Specifications.

Title: Round Column Design Using Strength Methods

Description:

Summary: The program calculates the maximum moment the column cross section can resist while supporting the design axial load, Pu.

Explanation: Maximum concrete strain is assumed to be 0.003. Reinforcing bar strain and concrete compression block size are varied (constrained by proportional strain relationships) until the total axial resistance of the concrete and steel together, Pn, reduced for column understrength, is equal to Pu. The understrength is calculated as ØPn. From these same bar and concrete forces, the moment resistance, Mn, is also calculated along with the understrength moment resistance, ØMn.

The routine calculates "Balanced Loading" by setting the bar strain at the extreme tensile side equal to the yield strain. The resulting axial and moment resistance is output as P(b) and M(b), respectively. Mb is the maximum nominal moment resistance of the section. The corresponding axial load, Pu = ØPb, is therefore the optimum loading that gives maximum moment resistance. More or less applied P load will decrease moment capacity. Therefore, it is unconservative to assume extra dead or other axial loading for PU < ØPb and conservative for Pu > ØPb.

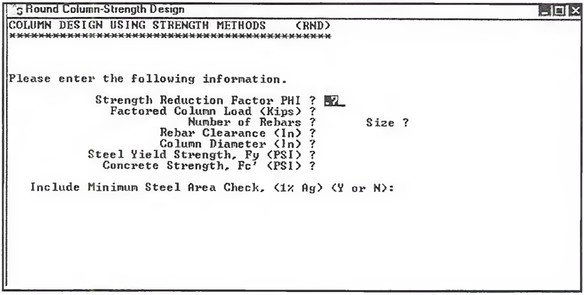
The moment Mn is calculated using the moment arms from the bars and concrete compression block to the geometric centroid of the column section. For axial loads that put the total cross section in compression, the centroid of the concrete force is conservatively assumed to be at the geometric centroid. (No moment contribution from concrete.)

The program gives a warning when the nominal axial resistance, Pn, required to resist the applied load, Pu, exceeds the absolute compression limit of the section as calculated by Section 8.16.4.1.2 of the AASHTO Standard Specifications, 16th Edition. This limiting value used by the program assumes a spiral-reinforced column. For tie-reinforced columns, the limit is 94% of the spiral value output by the program. (For typical bridge designs, this limit will rarely be of concern because axial loadings must be low enough to leave some capacity to resist the high moments imposed by lateral loading. Therefore, for the usual types of loadings, the axial load on a suitable bridge column is much less than the limiting value given by the program.)

The program calculates the area of longitudinal steel reinforcing from the number of bars specified by the user. If the area of steel is less than the minimum allowed by Section 8.18.1.2 (i.e., 1% of the gross column cross sectional area (Ag)), the program gives the user the option of trying the same load on a column section where the specified reinforcement will be equal to 1% of Ag. The reasoning is that, if a column with minimum steel area can support the required loads, adding extra concrete cannot diminish capacity. Similarly, if having minimum steel in a column prevents excessive creep strain in the steel, that strain will not increase by adding extra perimeter concrete. The reduced column uses reduced bar clearance equal to: (Actual Clearance) X (Reduced Diameter/Actual Diameter).

No flag is output for reinforcing ratios exceeding the 8% specified by Section 8.18.1.1.

Input:



Verification of Results:

Verify that the correct input values were used.

Title: Rectangular Column, Biaxial Bending – Strength Design

Location: Bridge Programs\Substructure

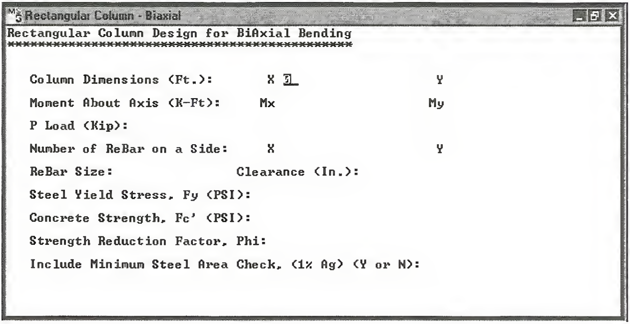
Description:

This program will investigate the adequacy of a square or rectangular column subject to uniaxial or biaxial loading based on the input provided by the user. In general, members subject to an axial load combined with bending are designed for the maximum moment that can accompany the axial load. This program calculates and compares the required eccentricity to the provided eccentricity. If the provided eccentricity is less than required, the program output will state to try a different design. Adjust the reinforcing steel and/or the column size until the provided eccentricity exceeds the required eccentricity.

Input:

This program uses customary U.S units only. Begin by double clicking the icon.

The following screen will pop up.

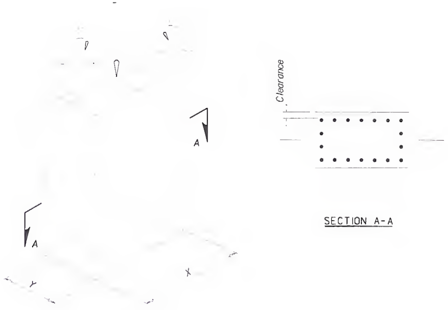


```
Rectangular Column - Biaxial

Rectangular Column Design for BiAxial Bending
*********************************************

   Column Dimensions (Ft.):        X 3_                Y
   Moment About Axis (K-Ft):     Mx                    My
   P Load (Kip):
   Number of ReBar on a Side:      X                   Y
   ReBar Size:                 Clearance (In.):
   Steel Yield Stress, Fy (PSI):
   Concrete Strength, Fc' (PSI):
   Strength Reduction Factor, Phi:
   Include Minimum Steel Area Check, (1% Ag) (Y or N):
```

For a description of the variables see the drawing below.



- **Mx** = Calculated design moment about the X axis.
- **My** = Calculated design moment about the Y axis.
- **P** = Calculated design axial load.
- **Number of Rebar on a side**: Select a number of bars per face.
  In the drawing above, X=7, Y=4
- **Rebar size**: Enter the bar size, 7 thru 18.

Note: Bridge Bureau policy states that the minimum number and size of bars per column is 8 #7 bars.

- **Clearance**: Distance from face of concrete surface to nearest edge of vertical steel. Default = 3.5 inches.
- **Steel Yield Stress, Fy (psi)**: Default = 60,000 (Grade 60)
- **Concrete Strength, F`c (psi)**: Default = 3000 (Class DD concrete)
- **Strength Reduction Factor**: Default = 0.7
- **Include Minimum Steel Area Check, (1% Ag) (Y or N)**: Default = Y.

Verification of Results:

Verify that the correct input values were used.

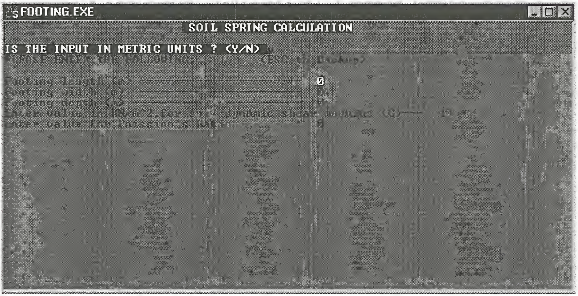
Title: Soil Spring and Footing Stiffness

Location: Bridge Programs\Other\footing.exe

Description:

Calculates equivalent radii for spread footings and for footings on piles, shape factors (α) and embedment factors (β) and footing stiffnesses about each of three axes for use in seismic design. (Method based on <u>*Seismic Design of Highway Bridge Foundations*</u>, FHWA Report Number FHWA/RD-86/101, Ignatius Po Lam and Geoffrey R. Martin, June 1986.)

Input:



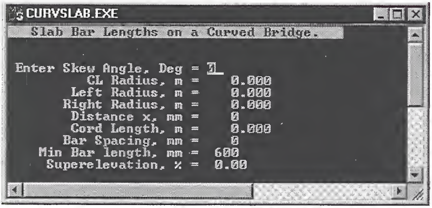
Verification:

Refer to the text to analyze output.

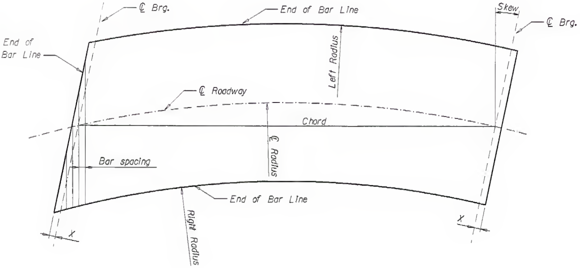
Title: Slab Bar Lengths on a Curved Bridge

Location: Bridge Programs\Metric\Curved Slab Bar Lengths

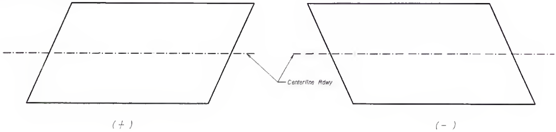
Input:



Graphical Description:



Skew Sign Convention:



Input Definitions:

Skew Angle: Measured from a perpendicular reference line to the bridge long chord generally unless there is an expansion joint in the slab (degrees). (See Skew Sign Convention)

Centerline Rdwy Radius: This program works for both right and left hand curves (meters).

Left Radius. This is the radius on the left side of the bridge that defines the end of the transverse slab bars at the edge of slab (meters). (0.075 meters from edge of slab)

Right Radius: Same as above except on the right side of the structure (meters).

Distance x: This point establishes first and last transverse slab bar. This distance is the perpendicular distance from centerline bearing to the end of the slab, and then subtract 40 mm clearance to establish the first bar position (mm). (Note: If x is not the same on both ends, run the program twice inputting first one side and then the next side.)

Chord Length: Chord length of continuous slab portion for which rebar lengths are being calculated (meters).

Bar Spacing: Centerline bar spacing of transverse bars measured perpendicular to bar layout (mm).

Minimum Bar Length: Default is 610 mm.

Superelevation: (%)

Output:

- Input values are displayed for verification.
- Bar lengths are given in skewed section of bridge on both ends.
- The units on the distance from centerline bearing of first bridge is shown as meters; however, the program prints the output in mm.
- Additionally, the program gives bar length difference between center of bridge and last full length bar before the skew at both ends. This is helpful to see if different bar lengths are needed.

Title: Skewed Slab Bar Lengths

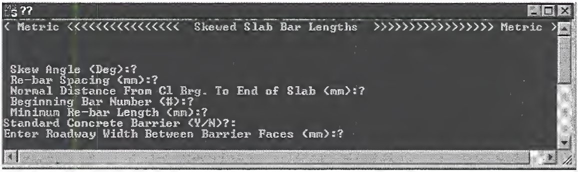
Location: Bridge Programs\Metric\Bar Lengths (Skewed Slab)

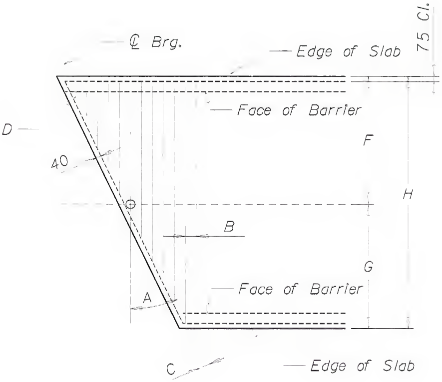
Description:

This program calculates the number and length of the variable length transverse slab bars in the skewed section of a tangent bridge.

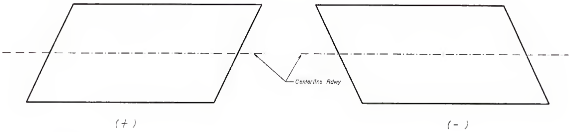
Input Screen:

The program will look similar to the screen shown below. (The program prompts for input one line at a time.)





Skew Sign Convention:



Input Definitions:

A - Skew Angle: (See diagram for sign convention)
B - Rebar Spacing: Centerline spacing of transverse reinforcement measured along chord
C - Perpendicular distance from centerline bearing to end of slab (see diagram)
D - Beginning Re-bar Number: Use bar numbers to match Bill of Reinforcing Steel
E - Minimum Re-bar Length: (Standard is 610 mm)

Standard Concrete Barrier?
If no, the program prompts for dimensions F and G.
If yes, input H.

Hints:

A peculiarity of the program is that, when standard concrete barrier rail is specified, the program only allows for input H. This does not allow for offset centerline of roadway. So when the bridge has an offset alignment with concrete barrier rail, input N when prompted for Standard Concrete Barrier. Then input dimensions F and G (centerline roadway to edge of slab).

Output:

Input values are displayed for verification. For the Standard Barrier Rail, the program calculates the dimension from centerline roadway to the slab edges based on the assumption that the centerline of roadway is centered between barriers.

The program calculates the lengths of each bar in the skewed section and assigns a bar number. In addition, the bar number and length of the rebar passing through the centerline of bearing is noted.

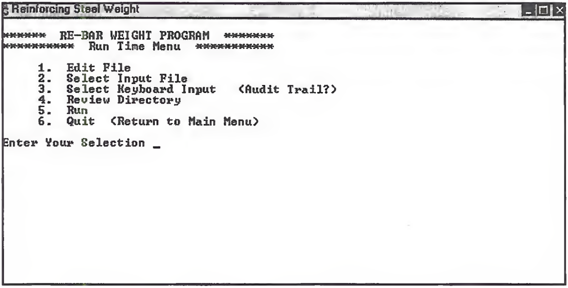
Title: Reinforcing Steel Weight

Location: Bridge Programs\Metric\Reinforcing Steel Weight

Description:

The program calculates and totals reinforcing steel weight for each bar size. Input the bar callout, the number of bars, the bar size and the length of the bar from the bill of steel. The program calculates the weight for each bar callout and the total weight for each bar size for Plain bars, Epoxy bars, Seismic bars or Epoxy-Seismic bars. This information is then entered into the Quantity Sheet and is used for estimating project costs by MDT and by the Contractors bidding the project.

Input:



Directions:

To start a new rebar run, select number 2 from the menu. Input filename consisting of the Control No. followed by the description and a .txt extension. Example: 1515Bt1.txt. The TXT file is usually saved in the same place as the program; in this case, it will be saved under the Bridge Programs in the metric folder.

Pick number 3 to input the data. On the first part of the bar mark, put the alphabetic coding for the bar (like C for a cap bar or BW for a backwall bar). Press the enter key to move to the next line. Enter the first bar number of the bar series, and press enter to move to the next line. Enter the last consecutive number of the bar series. Press the enter key to move to the next line.

Note: If there is a bar that is not in a consecutive series or if a bar mark is different from the rest of the bars, code the first and last bar the same.

The program will ask if the bars are epoxy coated or not; press Y if they are or press N if they are not. Follow the same process for seismic bars.

The next screen will ask for the bar size and the number of bars and the total length of the bar in millimeters. The program will go to the next bar in the run and ask for the bar size, number of bars and the total length of bar. Once the input for all bars in a series is complete, the program will return to the first screen and ask for a new alphabetic bar mark, and it will follow the same input above for each bar mark in the bill of reinforcing.

Once all of the bars are input, a summary screen will show the bar sizes and the total weight for that bar size for Plain, Epoxy, Seismic and Epoxy-Seismic.

Review the data prior to printing. If the information appears correct, print as indicated below. If corrections are needed, see directions below.

In the lower right corner of the next screen, the user will be prompted to print the output. Press Y if yes or press N if no.

The program will return to the beginning screen ready for the next bill of reinforcing to be input or, if the user is ready to quit the program, then select number 4.

If the data needs editing, go to notebook or other text editor. Open the TXT file that was created and make the changes in the TXT file and resave it. Go back into the Reinforcing steel weight program and select number 1. Type in the name of the file for which the changes were made to select number 3. It will give an overview of the bars. When done reviewing, press any key to move to the print screen. Press Y if yes or press N if no.

The program will return to the beginning screen ready for the next bill of reinforcing to be input or, if the user is ready to quite the program, then select number 4.

Verification of Results:

Verify that the correct input values were used by comparing the output sheets to each Bill of Reinforcing for the plans. Correct and rerun if needed. See Directions above.

Title: Prestressed Beam Diaphragm Volume

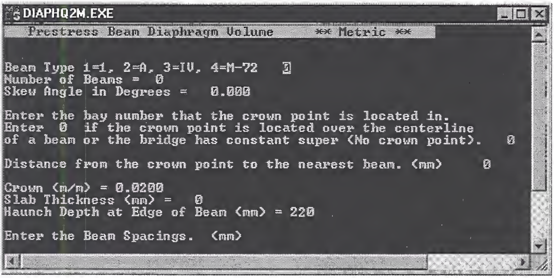
Location: Bridge Programs\Metric\Diaphq2m

Description:

This program calculates the concrete diaphragm volume for standard MDT girder shapes.

Input Screen:

The program will look similar to the screen shown below. (The program prompts for input one line at a time.)



Input Definitions:

The input for this program is explained in the above input screen.

Output:

Input values are displayed for verification.

The program computes Class DD concrete volumes in both end diaphragms and intermediate diaphragms. The volume of DD computed is to the bottom of the fillets shown on the SL-5 and SL-6 standard slab drawings. The program also calculates Class Special Deck concrete quantities above the bottom of the fillet and below the bottom of the slab. All quantities are for one line of diaphragms; therefore, the output will need to be multiplied by the number of diaphragm lines for each diaphragm type.

Title: End Bent Quantities

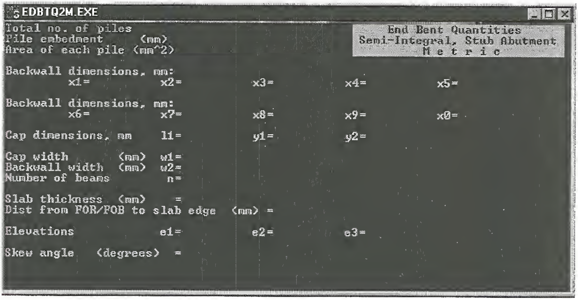
Location: Bridge Programs\Quantities\End Bent Quantities for English or Bridge Programs\Quantities/Metric\End Bent Quantities for Metric

Description:

This program calculates the volume of Special Deck and DD concrete in a typical stub end bent. Input the geometry of the end bent with elevations.

Input:

Below is what the program will look like on the screen.



Use the following Input sheets for ease of entry.

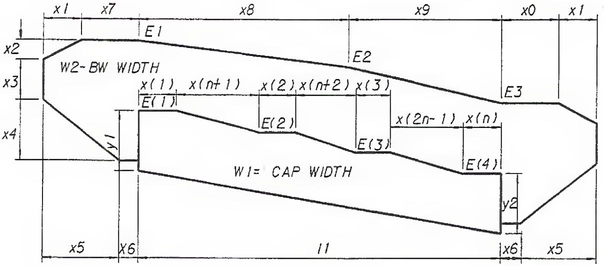
HINTS: For dimensions X6 through X0, use dimensions figured along the Fill face of the backwall.

When calculating volumes for an end bent using steel beams, enter any type of beam. No deductions are made for prestressed or steel beams.

Add the Special deck volume and paving notch volume back in for end bents used with Bulb T structures.

Add the paving notch volume to the Special Deck volume for end bents that do not have a
paving notch.

Stub Endbent Quantities Page 1 of 2



Total no. of piles		________	
Pile embedment	(Default 1)	________	ft
Area of each pile		________	ft^2
Backwall dimensions (x1-x5):			
		x1 = ________	ft
		x2 = ________	ft
	(Default 2)	x3 = ________	ft
		x4 = ________	ft
		x5 = ________	ft
Backwall dimensions (x6-x0):			
	(Default 1)	x6 = ________	ft
		x7 = ________	ft
		x8 = ________	ft
		x9 = ________	ft
		x0 = ________	ft
Cap dimensions:			
		l1 = ________	ft
		y1 = ________	ft
		y2 = ________	ft
Cap width	(Default 3)	w1 = ________	ft
Backwall width	(Default 1.5833)	w2 = ________	ft
Number of beams		n = ________	

```
                    Stub Endbent Quantities        Page 2 of 2

Slab thickness                               =________ in
Dist from FOR/FOB to slab edge               =________ ft  (default = 1.1667)

Elevations                                e1 =________
                                          e2 =________
                                          e3 =________

Skew angle                                   =________ degrees

Beam type(1=1, 2=A, 3=IV, 4=M-72, 5=BT-72, 6=OTHER) =________

For other types of beams:
    Slab thickness at edge of slab           =________ in
    Slab thickness at edge of ext. beam      =________ in
    Dist from ext. beam edge to slab edge    =________ ft

Horizontal length  &  elevation of one beam seat:
                                       x(1) =________ ft  e(1) =________
                                       x(2) =________ ft  e(2) =________
                                       x(3) =________ ft  e(3) =________
                                       x(4) =________ ft  e(4) =________
                                       x(5) =________ ft  e(5) =________
                                       x(6) =________ ft  e(6) =________
                                       x(7) =________ ft  e(7) =________
                                       x(8) =________ ft  e(8) =________
                                       x(9) =________ ft  e(9) =________
                                      x(10) =________ ft e(10) =________

Slope distance between beams:
                                     x(n+1) =________ ft
                                     x(n+2) =________ ft
                                     x(n+3) =________ ft
                                     x(n+4) =________ ft
                                     x(n+5) =________ ft
                                     x(n+6) =________ ft
                                     x(n+7) =________ ft
                                     x(n+8) =________ ft
                                     x(n+9) =________ ft
```

Verification of Results:

Verify that the correct input values were used.

Title: Area Centroid Moment of Inertia

Location: Bridge Programs\Geometry

Description:

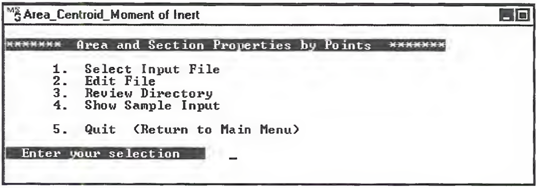
Program calculates the location of the centroid, moment of inertia, distance from neutral axis to extreme fiber, section moduli and radius of gryration about the x-axis and y-axis.

Input:

Start by creating an input file in a file editor program such as Notepad. See the example input file below. Save the file to C:\Programs\Geometry.

```
****************** Example Input File ********************************
9.5 -1.375       ' Enter one point per line.  Coordinate values may be
9.5,0            ' comma seperated or space seperated.
.3125 0
.3125 100        ' Comments may be placed on any line.  Comments are
8.35, 100        ' ignored.
8.35 101.125
sy               ' More then one set of data may be in one file.  Seperate
next             ' the date with the word  NEXT.
.3125  16.985
5.9875 16.985
5.9875 17.925
sy               ' SX and SY indicate that the section is symmetric about
sx               ' the X or Y axis.  The additional points are generated.
***********************************************************************
```

Save the file to C:\Programs\Geometry where the program will be able to read it.



Select the input file option. Type in the name of the input file created earlier.

Verification of Results:

Verify that the correct input values were used.

25.3 EXTERNAL PROGRAMS

See Figure 25.3A.

Name		Location
1.	AISIBEAM	Bridge Programs\Superstructure\AISIBEAM
2.	AISIsplice	Available on disk
3.	BRASS-GIRDER(LRFD)™	Programs\BRASS LRFD
4.	BT Beam	Bridge Programs\Substructure\BT Beam
5.	CONSPAN LA™	Bridge Programs\Superstructure\CONSPAN LA
6.	Florida Pier	Internet Download
7.	GEOPAK Bridge	Accessed through Microstation
8.	LPILE Plus	Bridge Programs\Other\LPILE Plus
9.	Mathcad®	Bridge Programs\Other\Mathcad
10.	MERLIN-DASH	Bridge Programs\Superstructure\Merlin Dash
11.	Opis	Desktop Icon
12.	Programmers File Editor (PFE)	Bridge Programs\Other\ Programmers File Editor
13.	Pontis	MDT Intranet
14.	SEISAB	Bridge Programs\Other\Seisab
15.	SIMON Systems	Available on disk
16.	System VANCK	Available on disk
17.	Virtis	Desktop Icon

EXTERNAL COMPUTER PROGRAMS

Figure 25.3A

Program Name: AISIBEAM

Location: Bridge Programs\Superstructure\AISIBEAM

Description:

The software is based on the Strength Design Method (Load Factor Design) of the AASHTO **Standard Specifications for Highway Bridges**, 16th Edition, 1996, including the 1997, 1998, and1999 Interim Specifications. The software uses Group I LFD load combinations. The Software will perform a line-girder analysis for simple-span girder and rolled beam bridges.

Inputs:

The following items can be customized by the user:

- Welded plate girders with unstiffened webs
- Rolled beams with optional welded cover plates
- Composite and non-composite stringers
- Normal weight and lightweight concrete deck material
- AASHTO strength, deflection, fatigue and constructibility criteria
- HS (MS) type truck load or special loading with up to 15 axle loads
- Up to 25 alternative solutions for each design
- Supporting design calculations
- Customary U.S. units or SI (metric units)
- Alternative fatigue load of HS (MS) configuration
- Operating and Inventory Ratings

Output:

In the design mode, the software will iterate between a range of user-specified minimum and maximum cross section dimensions to find a minimum-weight solution. In the rating mode, the user can input the exact cross section properties, and the Software will then solve for both an Inventory and Operating Rating.

Program Name: AISIsplice

Location: Available on disk

Description:

AISIsplice is a tool for the analysis and design of bolted field splices for straight, right, I-shaped, steel girders. The analysis and design process is based on the AASHTO **LRFD Bridge Design Specifications**, Second Edition, 1998, including the 1999 interim.

In the design mode, the software sizes and optimizes the splice plates and bolts. In the analysis mode, the software determines the adequacy of given splice plates and bolts. For both modes, performance ratios (load/resistance) for all splice components are determined.

Program Name: BRASS-GIRDER(LRFD)™

Location: Programs\BRASS LRFD

Description:

BRASS-GIRDER(LRFD)™ is a comprehensive system for the analysis of highway bridge girders. BRASS-GIRDER(LRFD)™ utilizes finite element theory of analysis and current AASHTO Load Resistance Factor Design (LRFD)Specifications.

Inputs:

BRASS-GIRDER(LRFD)™ uses Windows based Graphical User Interfaces (GUI) for data input. Input and output may be created using US Customary or SI Units.

System input is free format consisting of commands grouped logically to define the bridge structure, loads to be applied and the output desired.

Girder types may be simple span, continuous, hinged or cantilevered, with or without integral leg frame configuration. Girders may be constructed of steel, reinforced concrete or prestressed concrete (pre- and post-tensioned). Composite steel and composite prestressed concrete girders may be included. BRASS can analyze variable depth girders, such as tapered or parabolic. The user may specify (by name) predefined cross sections that are stored in the cross section library. The library contains nearly all AISC rolled wide flange shapes and most AASHTO standard shapes for prestressed concrete I-beams. Using a library utility program, the user may modify the geometry of the existing sections, add new sections or delete existing sections.

Stage construction may be modeled by respective cycles of the system for girder configuration and load application. Cycles are automatic if desired. The dead load of structure members is automatically calculated if desired. Additional distributed loads and point loads may be applied in groups and each group assigned to a specific construction stage. Distributed loads may be uniform or tapered and divided into sections to model sequential slab pours. Loads due to prestressing are calculated and applied internally. Live loads may be moving trucks or uniform lanes loads, which include the HL-93 vehicles described in the AASHTO LRFD Specifications. Impact may be user defined, as specified by AASHTO, or the user may reduce impact to model reduced speed limits.

Outputs:

The program computes moments, shears, axial forces, deflections and rotations caused by dead loads, live loads, settlements and temperature changes. These actions are utilized by various subroutines to analyze user-specified sections of the girder.

Program Name: BT Beam

Location: Bridge Programs\Substructure\BT Beam

Description:

BT Beam is an analysis engine for the analysis of simple and continuous girders in accordance with the AASHTO **LRFD Bridge Design Specifications**.

Inputs:

The input is a description of the girder in terms of geometry, boundary conditions, loading and code-compliance requirements.

Outputs:

BT Beam reports influence lines for each live load and then computes factored and distributed loads. Finally, load envelopes are reported. Output is ASCII and delimited table for importing into a spreadsheet.

Program Name: CONSPAN LA™

Location: Bridge Programs\Superstructure\CONSPAN LA

Description:

CONSPAN LA™ is a comprehensive program for the AASHTO LFD and LRFD design and analysis of simple-span and multiple-span bridges, constructed with prestressed precast concrete beams, in U.S. and SI units. CONSPAN LA™ incorporates both AASHTO Specifications into one interface. The user can design using one code, and simply toggle to the other code for quick and complete design comparisons. This feature makes the transition from AASHTO LFD Specifications to AASHTO LRFD Specifications simple and efficient.

Simple-span static analysis is performed for dead loads resisted by the precast sections. Continuous static analysis is performed for dead loads acting upon the composite structure. A continuous moving load analysis is performed for the live load.

Inputs:

CONSPAN LA™ makes the entry of project data convenient with a system of tab screens, dialog boxes, graphical button, menus and wizards. Designs are completed with CONSPAN LA's™ automated features.

Outputs:

CONSPAN LA™ presents analysis results in a variety of easy to view formats, from a one page design summary to comprehensive project reports. Analysis results and graphical sketches can be exported to spreadsheets and DXF Formats.

During individual beam designs, various deign parameters such as distribution factors, impact/dynamic allowance factors and allowable stresses are established. The strand and debonding/shielding patterns can be automatically generated by CONSPAN LA™ or specified by the user. Debonding constraints limiting the number of debonded strands can also be user-specified. Service load stress envelopes, generated by combining the results of the analysis, are checked against allowable limits. Factored positive moments and shears are checked against the ultimate strength capacity of the effective section. Mild reinforcement in the deck, at the piers, is computed for factored negative and positive moments. Many other code criteria such as cracking moments, horizontal shear, stresses at limit states, etc., are also automatically checked.

Program Name: Florida Pier

Location: Internet Download

Description:

The Florida Pier analysis program is a nonlinear finite element analysis program designed for analyzing bridge pier structures composed of nonlinear pier columns and cap supported on a linear pile cap and nonlinear piles/shafts with nonlinear soil. This analysis program couples nonlinear structural finite element analysis with nonlinear static soil models for axial, lateral and torsional soil behavior to provide a robust system of analysis for coupled bridge pier structures and foundation systems.

Inputs:

Florida Pier performs the generation of the finite element model internally given the geometric definition of the structure and foundation system as input graphically by the designer. This allows the engineer to work directly with the design parameters and lessens the bookkeeping necessary to create and interpret a model.

Outputs:

Florida Pier contains an analysis program, FLPIER, that is coupled with a graphical pre-processor FLPIER_GEN and post-processor FLPIER_PLOT. These programs allow the user of Florida Pier to view the structure while generating the model as well as view the resulting deflections, bi-axial and uni-axial interaction diagrams and internal forces in a graphical environment.

Program Name: GEOPAK Bridge

Location: Accessed through MicroStation

Description:

GEOPAK Bridge represents technology for object-oriented bridge design and bridge design project management, integrated with civil engineering. Offering a toolkit of bridge modeling options, the software can be used to model most types of bridge structures, from simple to complex.

GEOPAK Bridge is fully integrated with MicroStation and the GEOPAK Civil Engineering Suite software, enabling interactive modeling in a familiar environment while ensuring the necessary integration with other transportation design disciplines.

Inputs:

The software includes a Bridge Project Explorer that gives access to bridge components and their properties, stored in a single bridge project database.

The software includes a set of sample report templates for startup.

Outputs:

The software reports any failures and automatically adjusts the location of bearings and beam seats to comply with the user's haunch requirements.

A Reports Manager gives flexibility to custom design reports that meet the requirements of a project or organization.

The user can test deflection conditions using five different deflection methods. GEOPAK Bridge models its solid elements using Bentley's SmartSolid® with b-spline curves generated using Para Solids. Volumes and surface areas of concrete and steel are easily calculated.

Program Name: LPILE Plus

Location: Bridge Programs\Other\LPILE Plus

Description:

LPILE Plus (developed and marketed by Ensoft, Inc., Austin, TX) is a special purpose program based on rational procedures for analyzing a pile under lateral loading. Soil behavior is modeled with p-y curves internally generated by the computer program following published recommendations for various types of soils; alternatively, the user can manually introduce other p-y curves. Special procedures are programmed for developing p-y curves for layered soils and for rocks.

A single, user-friendly interface written for the Microsoft Windows© environment is provided for the preparation of input and analytical run and for the graphical observation of data contained in the output file. The program has been written in 32-bit programming codes for compatibility with the latest versions of the Microsoft Windows operating system. The program produces plain-text input and output files that may be observed and/or edited for their inclusion in project reports.

Inputs:

Several types of pile-head boundary conditions may be selected, and the properties of the pile can also vary as a function of depth.

Outputs:

The program computes deflection, shear, bending moment and soil response with respect to depth in nonlinear soils. Components of the stiffness matrix at the pile head may be computed internally by the program to help the users in superstructure analysis. Several pile lengths may be automatically checked by the program to help the user produce a design with an optimum pile penetration. LPILE Plus has capabilities to compute the ultimate-moment capacity of a pile's section and can provide design information for rebar arrangement. The user may optionally ask the program to generate and take into account nonlinear values of flexural stiffness (EI) which are generated internally based on specified pile dimensions, material properties and cracked/uncracked concrete behavior.

Program Name: Mathcad®

Location: Bridge Programs\Other\Mathcad

Description:

Built on an intuitive whiteboard interface, Mathcad collapses two traditionally separate processes — design formulation and documentation — into one. Mathcad functions as an interactive worksheet that utilizes standard math notation and equation entry to solve problems, so no programming is required. Its patented whiteboard interface immediately returns or updates results, eliminating manual recalculation work.

Inputs:

NA

Outputs:

NA

Program Name: MERLIN-DASH

Location: Bridge Programs\Superstructure\Merlin Dash

Description:

MERLIN-DASH was developed for use by bridge design engineers who function in a production environment. MERLIN-DASH was developed to offer a wide range of features and options to meet the demands of universal usage in the analysis, design and rating of steel and reinforced concrete bridges.

The structural analysis is performed via a series of modular subroutines based on the stiffness method.

Inputs:

A mesh generation capability allows for the incorporation of fully automated AASHTO Dead Load (DL) and Live Load (LL) sequences.

Outputs:

MERLIN-DASH incorporates a flexible sequence of operations initiated with analysis and proceeding, at the user's option, to perform any or all combinations of the following functions for the AASHTO WSD, LFD or LRFD methods:

- Analysis — A complete analysis for all AASHTO DL and LL conditions with recycling for changes in sections due to design.
- Design — Determination of the size of steel structural components based on a user-controlled design sequence leading to either minimum cost or weight.
- Code Check — Complete and detailed code check of all steel or reinforced concrete beam components, which reference specific AASHTO equation numbers and applicable coefficients.
- Rating — Detailed inventory and operating rating of all beam components using either the AASHTO live load provisions or special user specified vehicles.
- Staging — Dead load pouring sequence stage analysis.

Program Name: Opis

Location: Desktop Icon

Description:

Opis is AASHTOWare's next generation of bridge design software. BRASS-LRFD provides the system's structural analysis and specification checking engine.

Inputs:

Opis employs the same database and graphical user interface as Virtis.

Outputs:

Opis will provide a set of output reports to help the designer understand the performance of a new bridge. A tree-structured graphical representation of the LRFD Specifications indicate whether each article is passed or violated, and it provides access to the detailed calculations for the bridge and the specification text. A suite of X-Y plots shows moments, shears, deflections, actual vs. capacity envelopes, influence lines and other information. These will be incorporated into the report-writing feature currently under design.

Program Name: Programmers File Editor (PFE)

Location: Bridge Programs\Other\Programmers File Editor

Description:

PFE is a large-capacity, multi-file editor. Although it is primarily oriented towards program developers and contains features like the ability to run compilers and development applications, PFE also makes a very good general-purpose editor for any function. It has all of the standard editorial features — insert or overwrite mode; open files; insert one file into another; etc.

Inputs:

NA

Outputs:

NA

Program Name: Pontis

Location: MDT Intranet

Description:

Pontis is a comprehensive bridge management system developed as a tool to assist in the task of bridge management. Pontis stores bridge inventory and inspection data; formulates network-wide preservation and improvement policies for use in evaluating the needs of each bridge in a network; and makes recommendations for what projects to include in an agency's capital plan for deriving the maximum benefit from limited funds.

Inputs:

Pontis has been developed to provide the user with a well-organized and intuitive graphical user interface. The system consists of a set of modules, each of which has been designed to provide the user with the informational display, options and actions relevant to the module's particular function.

Pontis supports the entire bridge management cycle, allowing user input at every stage of the process. The system stores bridge inventories and records inspection data.

Outputs:

Once inspection data has been entered, Pontis can be used for maintenance tracking and Federal reporting. Pontis integrates the objectives of public safety and risk reduction, user convenience, and preservation of investment to produce budgetary, maintenance and program policies. Additionally, it provides a systematic procedure for the allocation of resources to the preservation and improvement of the bridges in a network. Pontis accomplishes this by considering both the costs and benefits of maintenance policies versus investments in improvements or replacements.

Program Name: SEISAB

Location: Bridge Programs\Other\Seisab

Description:

SEISAB was specifically developed for the seismic analysis of bridges. The overall objectives in developing SEISAB were to provide the practicing bridge engineer with a usable design tool and vehicle for implementing the latest seismic design methodologies into the bridge engineering profession.

Inputs:

Horizontal alignments composed of a combination of tangent and curved segments are described using alignment data taken directly from roadway plans. SEISAB has generating capabilities that will, with a minimum amount of input data, automatically provide a model consistent with the model currently being used to conduct dynamic analyses. Seismic loadings in the form of response spectra are stored in the system and may be easily referenced by the user. The central theme underlying the development of SEISAB was to provide the bridge designer with an effective means of user-program communication using a problem-oriented language developed specifically for the bridge engineer. User input data is thoroughly checked for syntax and consistency prior to conducting an analysis and numerous default values are assumed for the data not entered by the user.

Outputs:

SEISAB contains both the single mode and multi-mode response spectrum techniques included in the AASHTO **Standard Specifications for Highway Bridges** and in the **Seismic Design Guidelines for Highway Bridges**. SEISAB can be used to analyze simply supported or continuous deck, girder-type bridges with no practical limitation on the number of spans or the number of columns at a bent. In addition, earthquake restrainer units may be placed between adjacent structural segments.

Program Name: SIMON Systems

Location: Available on disk

Description:

SIMON Systems is a PC software system for the design of straight steel plate-girder (I- or box-girder) bridges. The designs are according to the 16^{th} Edition of the **Standard Specifications for Highway Bridges**. LRFD Specifications will be available in the future.

Inputs:

Program input can be made in either standard US or SI Units. For users who prefer a freeform screen-oriented input style to create SIMON input files, program SIP (Simon Input Processor) is also provided with SIMON Systems.

Outputs:

SIMON, the basic building block of SIMON Systems, is a line-girder program that will design up to 12 continuous spans that may contain hinges. The pre- and post-processing program SIMPLE is also provided as part of SIMON Systems. Program SIMPLE helps the user optimize the vertical web depth of an I- or box-girder.

Program Name: System VANCK

Location: Available on disk

Description:

System VANCK performs a V-Load analysis of a curved open-framed I-girder bridge system. VANCK then checks designated target girders in the cross section according to the provisions of the latest AASHTO **Guide Specifications for Horizontally Curved Highway Bridges**. The system is designed as a tool to simplify the preliminary design of curved steel I-girder bridges.

Inputs:

System VANCK utilizes input baseline stationing and cross-section information. It is limited to a minimum of two and a maximum of ten spans. In addition, the number of girders in the cross section must be at least two and not more than eight. Tangent segments and multiple radii of curvature are allowed but reverse curvature is not allowed.

Outputs:

In a single execution, System VANCK will determine the individual girder span lengths from input baseline stationing and cross-section information (program JINK), do a V-Load analysis of the curved-bridge systems (program VLOAD), and check the normal bending and tip stresses in the flanges and output fatigue stress ranges including the effects of lateral flange bending (program CURVCHK).

Program Name: Virtis

Location: Desktop Icon

Description:

Virtis is AASHTOWare's new product for bridge load rating, featuring state-of-the-art graphical tools to speed preparation of the data and application of the results. Using BRASS as its proven analytical engine for load factor rating, Virtis provides an integrated database where rating inputs and outputs can readily be stored, reviewed and re-used. Through this database and the application-independent GUI, a user may provide a 3-dimensional description of a bridge superstructure. This bridge data, then, can be used by a variety of line-girder, 2-D or 3-D analysis packages, permit/routing systems and other third-party produced applications.

Although Virtis is written in C++, its support of the industry-standard COM interface makes it possible to access the system's data and functionality from many commercial software packages, including Visual Basic®, Excel®, AutoCAD® and Microsoft Word®.

Inputs:

As the successor to BARS, Virtis can import existing BARS files.

Data can be provided in either cross-section or schedule-based forms for steel girder bridges and cross-section based only for reinforced concrete bridges and schedule based for prestressed concrete bridges at this time.

Virtis contains a host of features to make load rating as easy as possible. Libraries of standard vehicles, loads, steel and prestressed shapes, load and resistance factors, materials, parapets and other bridge components allow bridge models to be built quickly in a drag-and-drop manner. Using the Windows clipboard, all or part of a bridge can quickly be copied to another bridge. As a bridge model is constructed, a graphical schematic framing plan, elevation view, cross-section view and other schematics provide feedback and make common types of errors apparent.

Outputs:

Virtis provides flexure and shear ratings, computes dead loads and distribution factors if they are not manually input, and analyzes deteriorated sections.